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ELECTRIC WELDED STEEL PIPE LINES[1]

By F. W. Hanna[2]

Within the last decade electric welding of water, gas and oil steel pipes has greatly increased. This increase has been due largely to the saving of the steel required in other types of pipe for lap and joint efficiency. The East Bay Municipal Utility District has been using both the electric welded and the oxy-acetylene gas processes in its pipe lines for the past six years. Generally speaking, no difficulty has been found in using these processes in the manufacture of small pipes out of thin plates, but care has been found necessary in the manufacture of large pipes out of thick plates.

In building its Mokelumne pipe line as part of its Mokelumne River Project, the District constructed 82.7 miles of steel pipe line across the San Joaquin River Valley between the foothills of the Sierra Nevada Mountains and the Coast Range. The Mokelumne pipe line is a part of the East Bay Aqueduct, which comprises in addition thereto three tunnels 8 feet in diameter aggregating 5.7 miles in length and one tunnel and a reinforced concrete pipe each 9 feet in diameter, aggregating 7 miles in length. The East Bay Aqueduct is a part of the water supply system constructed by the District for nine cities lying on the eastern shore of the San Francisco Bay. On account of the diameter, length, plate thickness, and operating head of

[1] Presented before the California Section meeting, October 30, 1930.
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the Mokelumne pipe line, this discussion will be confined mainly to the description of the welding processes used in its construction.

The Mokelumne pipe line is composed of 58.6 miles of 65-, 13.9 miles of 63-, 9.1 miles of 61- and 1.1 miles of 54-inch internal diameter pipe made of steel plates of $\frac{3}{8}$-, $\frac{7}{16}$- and $\frac{1}{2}$-inch thicknesses. The working stress used in the design of the pipe was 15,000 pounds per square inch with a longitudinal joint efficiency of 90 percent; and the operating head ranges from 0 to 500 feet with a large amount near the higher limit. The most economical diameters and thicknesses for the heads involved were used, but inasmuch as the hydraulic gradient closely parallels the grade of the pipe for a large part of the length of the line, very little economy was obtained by this procedure. Of the total length of 82.7 miles of pipe, about 71 miles are buried underground with an average covering of about 3 feet; and the remaining 11 miles are supported on concrete piers, steel trestles and red wood bents.

Except on angles in the line, the pipe is made in 30-foot sections each composed of two steel plates joined by two longitudinal electric welds. The sections in about 74.4 miles of the line are joined together with riveted joints, those in 4.7 miles are joined by oxy-acetylene butt welded joints, those in about 2.1 miles by straight bump double fillet electric welded joints and those in about 1.5 miles by butt-strap double fillet electric welded joints. The joint originally specified is of the straight bump riveted type.

The steel used in the manufacture of the Mokelumne pipe line is of structural quality for forge welding, requiring a carbon content of not over 0.20 percent, a manganese content of between 0.35 and 0.60 percent, a phosphorus content of not over 0.06 percent, a sulphur content of not over 0.05 percent, a tensile strength of not less than 50,000 pounds per square inch, a yield of 0.5 of the tensile strength but in no case less than 27,000 pounds and an elongation in 8 inches of 25 percent. The specifications also require that the steel shall preferably be free from silicon, nickle and chromium and that in no event shall the maximum quantity of any one of these elements exceed 0.05 percent. The steel plates were rolled by the Carnegie Steel Mills in Pennsylvania, shipped by rail to Newport News, Virginia, and there sheared to proper dimensions by the Newport News Shipbuilding Company then shipped by water to Berkeley, California, to the plant of the California Steel Tank and Pipe Company, where they were manufactured into pipe sections. The plates as

received were passed through 30-foot rolls, assembled into sections and tack welded at 3-foot intervals along each seam and then welded along these seams in Lincoln automatic welding machines.

The welding was done inside the pipe section on the bottom seam first and the section was then rotated 180° into position for welding the second seam. The welding head carrying the carbon electrode traveled on a channel-iron beam along the seam to be welded with the carbon electrode in a vertical position over it; and the electric arc between this electrode and the pipe plates fused the edges together.

An inside reinforcing bead was produced by a $\frac{5}{16}$- by $\frac{5}{8}$-inch filler rod, laid loosely on top of the seam, which was fused with the plates at their edges. The width of this bead was determined by confining the molten metal between two water-cooled copper bars or "fire strips" about $\frac{3}{4}$ inch apart closely pressed against the plates and extending the full length of the longitudinal seam.

A smaller outside bead was formed by a semi-circular groove in a water-cooled copper bar pressed firmly against the outside of the plates along the line of the longitudinal seam. The fused metal flowed into this groove as the arc penetrated the seam from above.

As the welding head progressed through the section the operator traveled through it behind the head, regulated the speed, controlled the length of arc, and adjusted the position of the electrode by means of devices carried on the welding head.

The speed varied with the thickness and composition of the plates and filler rods. With $\frac{1}{2}$-inch plates and filler rods $\frac{5}{16}$ inch thick, a suitable speed was about 12 feet per hour, but speeds of 14 to 16 feet per hour sometimes seemed to give equally good results. The current in the arc circuit ranged from 500 to 600 amperes, being somewhat greater for the faster speeds; and the voltage at machine terminals was about 44 to 48, which included the drop in the leads and the arc. Initially the carbon electrode was positive, but later the plate metal was made the positive terminal.

Many variable factors other than the skill and experience of the operator entered into the problem, one of which was the composition of the filler rods. The use of rods of the following chemical composition gave better results than ordinary rods: carbon, 0.12 to 0.17 percent; manganese, 0.35 to 0.54 percent; phosphorus, 0.01 to 0.02 percent; sulfur, 0.026 to 0.036 percent; vanadium, 0.15 to 0.30 percent.

The vanadium content in these rods was found to reduce the gas

pockets and slag inclusions, but the results were variable with different lots, due apparently to the use in manufacture of steel scrap comprising various alloys. One lot of these so-called vanadium rods gave trouble from the start and finally had to be discarded. The chemical contents of composite samples from these rods indicated the following composition: carbon, 0.16 percent; manganese, 0.60 percent; phosphorus, 0.016 percent; sulfur, 0.019 percent; nickel, 0.14 percent; silicon, 0.15 percent; chromium, 0.12 percent; vanadium, 0.16 percent.

Apparently the last four elements named above were badly segregated in the various rods since a wide variation was found by tests of numerous samples. The effect in the welding operations was to render it extremely difficult at times for the arc to penetrate the filler rods and fuse the plate metal and produce the required outside bead. It was thought that this effect was due chiefly to the presence of chromium. In order to secure proper penetration the operator had to reduce the speed of the welding head, which resulted in excessive boiling of the fused metal and weak welds with large gas and slag content.

Aside from these irregularities, better welds were produced with vanadium filler rods than with a great variety of other kinds, the vanadium acting as a scavenger to reduce the oxides in the molten metal. Analysis of drillings from welds made with vanadium rods indicated that the metal in the welds retained only about a third of the carbon contained in the original metal of the filler rods and pipe plates.

Other means tried for reducing the oxides consisted in covering the filler rod with powdered glass, or borax glass, to a depth from $\frac{1}{8}$ to $\frac{1}{4}$ inch, which fused and covered the puddle of molten metal and kept the air out. Mechanical reasons made these attempts unsuccessful as they also did efforts to use flux coated rods.

One of the principal causes of bad welds was the tendency of the arc to move persistently to one side of the seam, resulting in the weld being thrown partly off the center line of the seam and the fusing of the copper from the "fire strips" into the weld. This instability of the arc was due in part to magnetic forces set up by the welding current and to the inability of the portable magnet in use to counteract the magnetic forces. This difficulty has undoubtedly been overcome in the later welding machines by the use of a solenoid on the welding head.

After the longitudinal seams were welded, they were carefully inspected for defects, and the defective welds were chipped down to good metal and repaired by electric hand welding outfits. The pipe sections were then tested under hydraulic pressure 50 percent greater than the specified working stress. If any leaks were found in a section, they were repaired and the section again tested. While under pressure the pipe was subjected to thirty blows along each seam by trip hammers, of a specified weight, suspended from above. The tested sections were thoroughly scoured of scale and rust by revolving brushes, then placed in a furnace, held there until the temperature of the sections reached about 450°F., removed and vigorously hammered to remove adhering scale or dirt, dipped vertically into a pit filled with a melted asphaltic-tar compound, cooled and finally wrapped in an automatic wrapping machine with a heavy strip of felt adhered with melted asphalt.

The specifications calling proposals on the Mokelumne Pipe Line provided for straight bump riveted circular seams, but gas and electric welding were contingently provided for in the contract. Owing to the fact that serious tensile stresses were developed in the pipe by the gas welding process due to welding heat, combined with daily temperature changes, the bulk of the circular seams were riveted. However, as already stated, the sections in about 8.3 miles, or one-tenth of the entire line, were joined together by the gas and electric welding processes.

In the electric welded seams two types of joints were used, the butt-strap type and the straight bump type. In the butt-strap type, the butt straps were fillet welded at the edges of the butt straps and at the edges of the pipe sections, making four electric fillet welds at each joint. In the straight bump type, the sections were joined by two fillet welds, one inside and the other outside of the pipe. This method gave highly satisfactory results.

Early in the construction of the Mokelumne pipe line the writer proposed a definite procedure for welding the circular seams and this procedure was followed in the successfully electric welded joints just described.

In the straight bump type of joint the specified procedure required that the spigot end of each section of the pipe be placed in the bell end of the preceding section and brought tightly home by suitable means, that the pipe be laid in stretches not exceeding 690 feet in length to be joined together at night with quadruple fillet welded butt strap

joints, that the outside welding of the straight bump joints be done between the hours of 7 p.m. and 8 a.m. by two welders working simultaneously progressing in the following sequence: (*a*) about 30° downward each way from the top of the pipe, (*b*) about 20° upward each way from points 10° below the ends of the horizontal diameter, (*c*) about 80° upward from the lower end of the vertical diameter to the points already welded, and (*d*) about 50° upward to fill in the gaps between the first and second welded portions, that the inside fillet weld be made during daylight hours by two welders each following the same sequence and finishing before 7 p.m., and that the pipe be partially backfilled as soon as practicable after being placed in the trench. The pipe was installed in 690-foot stretches so connections between them could be made at night when the pipe was contracted by the drop in diurnal temperature; the specified sequence and time of welding were used to distribute the welding heat around the joints and to profit by the uniform, circumferential temperature of the pipe at night.

The procedure prescribed for the butt-strap electric welded joints also required that the pipe be laid in stretches not exceeding 690 feet in length to be joined together with quadruple fillet welded butt-strap joints similar to those used for the straight bump joints and at night for the same reasons. In joining the sections in each group length, semi-circular butt straps tightly cinched to the pipe were used and the lower half of the butt strap on the forward end of each section was double fillet welded before the section was placed in the trench by one welder welding first one quadrant then the other. After a pipe section was placed in the trench, the remaining half of the butt strap on the preceding section was tightly cinched into place and welded in a manner similar to that used for the first quadrant. The quadrants of the butt straps were then welded together at the ends with V butt welds. The inside and outside fillet welds on the rear end of the forward section were then completed by two welders in the sequence and the hours prescribed for straight bump joints. The pipe was semi-backfilled as soon as possible after it was placed in the trench.

As already stated, the oxy-acetylene gas process was applied to about 4.7 miles of the Mokelumne pipe. In this process an outside 60°V. butt weld was used; but the process proved unsatisfactory. The welds cracked at night, mainly at the top of the joints, and attempts to reinforce the outside welds with inside electric welds

were unsuccessful. These failures were partly due to the use of the butt type of joint and partly to sequence and time of doing the welding.

It is difficult in a pipe of large diameter composed of thick plates to form a perfect butt weld in the field either by the gas or electric process, because differences in temperature between the top and bottom of the pipe produced either by diurnal changes or welding heat make it impossible to secure a weld of uniform thickness. Moreover, the welding was done in the daytime when there was as much as 70° difference in the normal temperature at the top and that at the bottom of the pipe; and, as the welding was done by beginning at the bottom of the pipe and proceeding to the top the welding heat augmented this difference in temperature. Consequently the width of the weld at the top was narrower than at the bottom of the pipe, and, in some instances, it was necessary to cut out some of the plate metal at the top in order to make an opening for the weld. At night, when the temperatures of the top and the bottom of the pipe became more nearly equal, the greater contraction on the top naturally produced tensile stresses and cracks in the welds. Consequently the oxy-acetylene process was abandoned, and the joints already made were strengthened by electrically welding to the pipe reinforcing plates 12 inches long in the direction of the axis and 15 inches wide in the direction of the circumference of the pipe. One plate was placed with its center at the top, two with their centers at 60° from the top, and two more with their centers at 60° below these. These plates made the joints entirely safe and satisfactory, but two expensive for use on new work. Undoubtedly had a butt strap or straight bump type of joint been used and had the sequence and time of making the welds been made to conform to those prescribed for the electrically welded joints, the acetylene process would have been successful.

The East Bay Aqueduct has been in successful operation since June, 1929. There are 870,000 linear feet of electrically welded longitudinal seams and 24,000 linear feet of circular seams in the 82.5 miles of steel pipe line. In this length there have so far been a total of 290 leaks. Of these leaks over 50 percent have been mere pin holes and the remainder have mainly been small holes varying from $\frac{1}{16}$ to $\frac{3}{4}$ inch in diameter in the weld metal. The total length of defective welds thus far developed have amounted to only 10 feet out of the total of 894,000 feet. The majority of the leaks have been in the

$\frac{3}{8}$-inch plate pipe. There have been 18 cross cracks varying from 3 to 11 inches in length and 3 splits along the welded seams, one 6 inches, another 11 inches and another 60 inches in length. All of the cross cracks and splits have occurred in the $\frac{1}{2}$-inch plate pipe. For the large diameter of the pipe, the thickness of the steel plates, the length of the line and newness of the application of the welding process to this type of pipe, it is considered that the results are very satisfactory.

WATER SUPPLY PROBLEMS IN OHIO OCCASIONED BY THE DROUTH[1]

By F. H. Waring[2]

The year 1930 goes down in history as the driest year on record in Ohio. A similar statement will probably be applicable to at least 20 other states. Weather Bureau records show that the southern portion of the state was more seriously affected than the central or the northern portions. Rainfall records show that the deficiency for the year for the southern portion was from 15 to 19 inches, as compared with an average annual rainfall of 39 inches; for the central section the deficiency was from 9 to 15 inches, as compared with an average annual rainfall of 36 inches; in the northern portion the deficiency varied from normal to 9 inches, as compared with an average annual rainfall of about 33 inches.

The drouth in Ohio began about the first of April and continued throughout the remainder of the year. As a result water supply problems were experienced in many places throughout the entire state. Such problems were most acute in the southern section. Surface water supplies were decidedly affected. Ground water supplies are only now beginning to experience difficulties. Surface water supplies depend upon rainfall and runoff throughout the greater part of an entire year, whereas ground water supplies depend largely upon the percolation of the water into the ground during the early months of each year. Consequently the surface water supplies have been seriously embarrassed for several months, whereas ground water supplies have up to now shown little or no shortage difficulties. There are exceptions to the foregoing statements depending upon the local conditions of water supply development.

RECENT SURVEY OF HEALTH HAZARDS IN DROUTH AREA

During the past few weeks the Ohio Health Department has made an intensive study of the public water supplies in the drouth area in

[1] Presented before the Indiana Section meeting, February 26, 1931.

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southern and eastern Ohio embracing 39 counties and 115 public water supply systems. In addition the department has surveyed the rural health hazards in these counties. The public water supply studies were conducted by the division of sanitary engineering, using from 5 to 9 assistant engineers in the field during the three weeks period. The rural health hazard surveys were conducted by three two-man teams of physicians and sanitary engineers, assisted by the public health nursing staff of the department.

Some of the general conclusions to be drawn from the studies recently completed were the following: (1) Public water supplies obtained from ground water sources have not been seriously embarrassed. (2) Public water supplies obtained from surface sources have been embarrassed, (*a*) by water shortage difficulties, (*b*) by disagreeable tastes and unusual hardness characteristics. (3) Rural water supplies from individual wells, such as serve the crossroads communities and the farmers, have been depleted almost entirely. (4) Sanitary conditions have continued good in spite of the drouth, as shown by the fact that this area is unusually free from the prevalence of communicable diseases of all kinds. (5) Economic conditions are bad and have been accentuated by reason of the drouth; for example, supplies of food, clothing and shelter have been found inadequate to take care of ordinary needs in a very considerable portion of the drouth area.

CLASSIFICATION OF THE PUBLIC WATER SUPPLIES IN THE DROUTH AREA

Problems of public water supply brought on by the drouth are varied, and consideration will be given to the character of these problems, as well as the relief that was possible, according to the classification of the supply.

Of the 115 public water supplies in the 39 counties served, 94 are obtained from well sources and 21 from surface sources. The 94 well water supplies are further classified into 71 with no treatment, 12 with chlorination, 8 with softening, 3 with iron removal. The 21 surface supplies are classified into 19 with filtration devices and 2 without any treatment devices.

Considering the surface water supplies first, it is to be noted that 8 of them are derived from the Ohio River. These supplies are purified by means of the usual rapid sand filtration type of plant. The problems during the past few months at these 8 Ohio River cities are briefly described as follows.

PROBLEMS OF WATER SUPPLY COMMON TO COMMUNITIES ON OHIO RIVER

Tastes. By reason of the long continued drouth a taste characteristic was acquired by Ohio River and tributary streams variously described as musty, moldy, burnt leather, and commonly referred to as "river taste." Undoubtedly the long continued pool stage of the river, with consequent stagnation of decomposing organic and vegetable matter supplemented by industrial wastes, combined to form this river taste. A study of the experiences at several filtration plants reveals some interesting comments and with respect to coping with the taste characteristic.

a. The employment of excess lime as in water softening served greatly to minimize the river taste, as shown by the experiences at Bellaire and Ironton. These tastes were further minimized when chlorination was suspended in conjunction with the excess lime treatment (interruption of excess lime treatment at Bellaire in December brought forth taste complaints; increase in lime treatment and discontinuance of chlorination at Cincinnati relieved taste complaints).

b. Potassium permanganate was employed at one plant successfully for three days (Delaware, Ohio, located on Olentangy River, a tributary to the Ohio River system). About 3 pounds per million gallons were applied at a cost of 16 cents per pound or 48 cents per million gallons. It is necessary that such treatment be followed by lime treatment in sufficient quantity to precipitate the manganese, the suggested point of application for the permanganate being into the raw water at the beginning of the mixing chamber, the lime being added halfway through the mixing chamber. The amount of lime needed to precipitate the manganese is assured if the alkalinity of the water treated approaches causticity. Bottle experiments upon river water elsewhere in the drouth area have shown equally satisfactory performance, but no other plant scale experience was had.

c. The ammonia-chlorine process was used at Warren, East Liverpool, Marietta, Pomeroy (and recently at Cincinnati). The experiences at these places show that the ammonia-chlorine process did not prevent nor modify the characteristic river taste. It did serve, however, to prevent phenolic tastes. (East Liverpool, Marietta and Pomeroy did not have phenolic tastes in December and January when other places along Ohio River experienced them.) It was observed that great difficulty was experienced in maintaining residual chlorine because of the frequent high organic content of the river water, which content varied from day to day. On one day Warren

(Mahoning River) was unable to obtain a residual chlorine using 7 pounds of chlorine per million gallons. A similar experience was said to have existed at Pittsburgh (Allegheny River) for a day or two during the drouth period.

d. Activated carbon has been used at the Huntington, W. Va., plant which also serves Chesapeake, Ohio. Experience with continuous carbon treatment during the drouth period has revealed that the water so treated is free from the river taste. A similar experience is reported from New Castle and South Pittsburgh, Pa., at plants owned and operated by the same company that operates the Huntington plant. The treatment being practiced is the application of "Nuchar" in the amount of one-third grain per gallon or 50 pounds per million gallons. This material sells at about 8 cents per pound in commercial quantities, making the cost of such treatment about $4.00 per million gallons. Activated carbon may be procured from several companies, among them being the Darco Sales Corp., 45 E. 42nd St., New York City, and the Industrial Chemical Sales Corp., 230 Park Ave., New York City. The material is applied from dry feed machines to the raw water in the mixing chamber ahead of the coagulants. Probably an ideal arrangement would be to add the carbon to the water after coagulation, if a mixing chamber were arranged between the settling basins and the filters; in this manner the carbon would be carried over onto the top of the sand filters and further serve by forming a thin layer of carbon filter.

Hardness. Another criticism against the Ohio River during the past few months on the part of the water users has been its excessive hardness. For example, the average total hardness of Ohio River water at Cincinnati in 1929 was 100 p.p.m., of which 39 p.p.m. was alkalinity or temporary hardness and 61 p.p.m. was incrustants or permanent hardness. From September, 1930, to January, 1931, inclusive, the average figures for Cincinnati by months are as follows:

MONTH	PARTS PER MILLION		
	Alkalinity or temporary hardness	Incrustants	Total hardness
September, 1930	59	99	158
October, 1930	71	75	146
November, 1930	67	101	168
December, 1930	58	174	232
January, 1931	49	136	185

Attention is invited to the December figures when the average total hardness for the month was 232 p.p.m.—the maximum ever recorded at Cincinnati since the plant was placed in service in November, 1907.

The attempt was made during December at the Cincinnati plant to reduce the hardness somewhat by extra lime treatment. Owing to limitations of plant layout and facilities it was not possible to apply more than 3 grains of lime per gallon. This amount, however, served to reduce the total hardness by 30 p.p.m. Other plants along the river made comparatively little or no attempt to reduce the hardness to normal figures.

Sickness. A serious criticism against the Ohio River water was registered in December and January just past. A widespread prevalence of sickness was observed in Cincinnati and other river cities, the ailment being designated commonly as "intestinal flu." Because of the bad tastes in the city water supplies of the Ohio River cities, the people, and some physicians, were quick to blame the city water as being responsible for the ailment. It should be stated that in all of these Ohio River communities the quality of the water supply as served to the consumers was safe as measured by bacteriological standards. Two theories are being advanced by medical men conversant with the situation. (*a*) The prevalence of a type of influenza especially affecting the intestinal tract. (*b*) The existence of a toxic property in the river water by reason of the decomposition of organic substances of vegetable and animal origin.

It may never be known which of the foregoing theories was correct or what part each may have played in the prevalence of sickness. It is sufficient to say that the popular belief was that the bad taste of the city water was associated with the sickness. Such a situation is reason enough for the expenditure of considerable funds by the city water department to eliminate the bad taste, either by way of purchase and installation of equipment, or by purchase and use of additional chemicals in the water treatment.

IMPORTANCE OF WATER WORKS PERSONNEL IN WATER SHORTAGE SITUATIONS

Some interesting experiences were recorded at some of the cities using surface water supplies not taken from Ohio River. These experiences indicate that great dependence must be placed upon the personnel in charge of water works departments. We have found

it to be the case that *personnel alone* was responsible for two serious water shortage situations in Ohio cities, and that *personnel alone* was responsible for avoidance of water shortage situations in one other Ohio city, all of them being in the drouth area not very many miles from each other.

1. Barnesville with a population of about 5000 takes its water supply from a surface impounding reservoir upon a small watershed. A modern filtration plant purifies the water. Failure to conserve the supply throughout the summer and fall months by enforcing the ordinary restriction measures resulted in the supply being practically depleted in December. For a month thereafter water was drawn from the pocket of the reservoir which should have been held for fire reserve. The water consumption averaged over 400,000 gallons per day in September and October, but when the situation became acute the supply consumed registered less than 200,000 gallons per day, showing the conservation that could have been, but was not, practised during the months previous. Emergency measures adopted were the construction of two temporary impounding dams upon adjacent watersheds, and the pumping of runoff during January through pipe lines two miles long against a static lift of 300 feet to get the water into the regular impounding reservoir. Emergency wells afforded little or no relief.

2. Woodsfield, with a population of 2500, relies upon a similar impounding reservoir on a small drainage area for its storage of raw water supplying the filtration plant. The supply was depleted to less than five days during January. Emergency wells yielded sufficient water to carry the village until the light rains and runoff of early January occurred. An adjacent creek then afforded sufficient runoff to pump direct to the filtration plant. An argument ensued between the water company and the municipal electric light officials over the contract for electric service for the emergency pumping, as a result of which the water company ceased pumping water from the creek direct, thus jeopardizing the safety of the community with less than five days storage of water on hand. The amount of electric current involved represented about $230 expenditure for one month's operation of the emergency pump. The rains of February have been steadily replenishing the surface reservoir but the emergency pump now idle could have long since filled the reservoir and have eliminated any possible danger of water shortage for the present and immediate future. Both city and company officials are to blame

for the situation; and should any disaster occur at Woodsfield, both, in the writer's opinion, would be responsible.

3. Cambridge, with a population of about 15,000 and located in the same general vicinity, took care of itself and adjacent territory, including industrial demands by careful planning and execution of certain expedients. The water supply, ordinarily obtained from Wills Creek and pumped to a storage reservoir and thence by gravity to the filtration plant, was augmented by the pumping of water from abandoned mines over a period of almost four months. It became necessary from time to time to quit pumping from certain mines and start up from other mines up the valley on account of the salinity and acidity characteristics of the mine water being pumped. Additional efforts were being planned when the rains of January relieved the situation and obviated the necessity of further mine water pumping. Today Cambridge's reservoir is full and the shortage crisis has passed. Throughout the entire period careful operation at the filtration plant kept the treated water of uniform quality, irrespective of the acidity, iron and hardness characteristics of the mine waters being received. The salt content of the water was the real factor that caused the abandonment of certain of the mine water sources of supply. The water works personnel at Cambridge is to be congratulated upon their excellent management during the water shortage crisis.

Numerous other instances of both mismanagement and good management could be recited with respect to the 14 surface water supplies in the area, not including the Ohio River supplies. Time will not permit to discuss more of these experiences.

EFFECT OF DROUTH UPON GROUND WATER SUPPLIES

With respect to the experiences had with the 94 well water supplies in the drouth area, it is sufficient to say that perhaps less than a dozen have experienced any effect whatever at present writing. Even in these relatively few municipalities, relief was obtained by the drilling of an additional well or two. Most of them, however, have observed a lowering of ground water level. If the original development of the ground water resources has been proper the immediate lowering of ground water level has not been a serious item. What the situation may be within the next few months it is difficult to predict. From now until June the rainfall will have to be copious, and the ground will have to be porous, if the ground water levels are not to be lowered

over a considerable period of time. While such a lowering of level may not be permanent, in a sense it is permanent if the municipality is faced with the necessity of pumping from lowered ground water level over a period of a few years.

CONCLUSION

The experiences in general have served to accentuate the improvement programs of water systems all over the state. Some systems experienced only distribution difficulties, some only pumping difficulties and some had storage difficulties. It is safe to say that there will be considerable activity on the part of municipalities and water companies during the ensuing year toward the improvement of water systems as result of the lessons taught by the drouth of 1930.

One comment is pertinent at this time, as a ray of sunshine in the gloom of financial depression which has been coincident with the drouth experienced. Business depression has resulted in less industrial demand of water. The drouth caused more domestic demand of water. The two have about balanced each other to make a normal water demand during the year. In other words, without the industrial depression the water shortage problem would have been much more pronounced; and acute situations that did not develop would certainly have developed. As the saying goes, "it is an ill wind that blows nobody good."

TYING TOGETHER LARGE CAST IRON PIPE AND FITTINGS BY BANDS

By William W. Brush[1]

For many years it has been the practice in New York to use bands and bolts to hold pipe from pulling apart where bends, offsets, tees, reducers or caps are used and there are no lugs on the bell or spigot end of the pipe and fittings. In special cases such as the high pressure installations of recent years an amalgam of tin and lead has been successfully employed, but usually the bolts and bands are installed. For a 90° bend it is customary to tie together pipes that are 12 inches and larger in diameter. On each side of a 90° bend normally three lengths of 12-inch pipe are rodded while for 20-inch and larger pipe five lengths are rodded. Where other than a 90° fitting is used the number of lengths to be rodded is determined by the construction engineer based upon pressures in the pipe and character of fitting as affecting the amount of unbalanced pressure to be taken up by the rods.

Our experience and that of others indicate that in a caulked lead joint the holding power for each square inch of lead in contact with the pipe is about 200 pounds. When the unbalanced pressure produces a force to separate the pipe greater than this holding power, the pipe moves in the joint. Under such conditions the joint must be held together by some other means.

In November, 1927, the department was compelled to raise a 48-inch cast iron main laid in Columbia Street, Queens, due to the main interfering with a new sewer. It was decided to offset the main after reducing to 36-inch diameter on account of limited cover. The standard department band is shown in figure 1, and figures 2 and 3 show details of the construction of the pipe offset. When a pressure of about 80 pounds per square inch was applied the band on the spigot end of one 48-inch pipe moved about $\frac{1}{4}$ inch and the offset raised in the center sufficiently to allow the lower rods to drop out of

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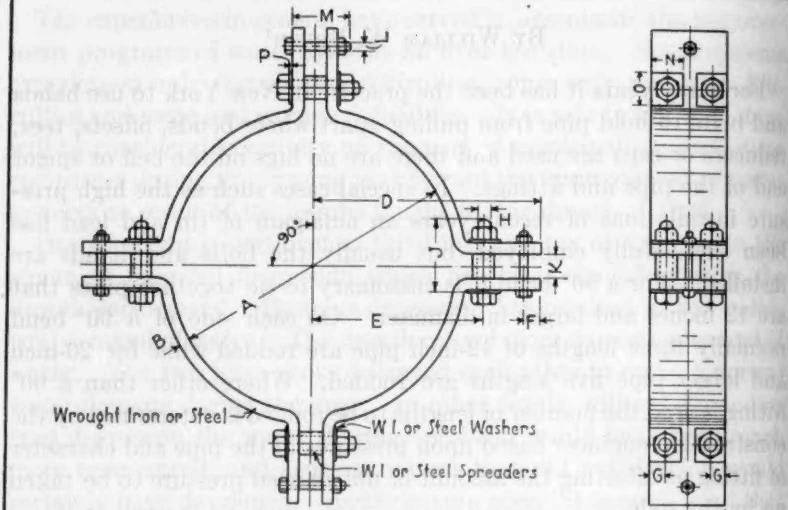
place. From this experience it was evident that the bands used would not hold unless stud bolts or similar stops were placed in the pipe or unless rods were carried back of the next bell. The latter plan was considered more advantageous and was adopted as standard for future construction where lugs are not available. With all bands



All holes drilled ⅛" larger than diameter of bolt

Size of Pipe	A	B	C	D	E	F	G	Main Bolts No.	Main Bolts H	Main Bolts J	Small Bolts No.	Small Bolts L	Small Bolts M	Spreader Inside Diam.	Spreader K	Washer N	Washer O	Washer P	Weight Without Bolts	Weight Complete
4"	4⅞"	⅜"	3"	7½"	4⅜"	1"	1½"	4	⅞"	3½"	4	⅝	3	1"	1¼"	2"	2"	¼"	15*	25*
6"	7	⅜	3	8½	5⅜	1	1½	4	⅞	3½	4	⅝	3	1	1¼	2	2	¼	17	27
8"	9	⅜	3½	9½	6⅜	1	1¾	4	⅞	3½	4	⅝	3	1	1¼	2	2	¼	22	32
12"	13⅜	½	4	12¼	8⅞	1	1	8	⅞	4	4	⅝	3½	1	1½	2	2	¼	43	62
16"	17¼	½	4½	14¼	10⅞	1	1	8	⅞	4	4	⅝	3½	1	1½	2	2	¼	55	74
20"	21½	½	5	16⅜	13	1	1¼	8	⅞	4	4	⅝	3½	1	1½	2	2	¼	64	83
24"	26	½	5	19	15¼	1	1¼	8	⅞	4½	4	⅝	3½	1	1⅞	2	2	¼	82	102
30"	32¼	⅝	5½	22⅞	18¾	1	1¼	8	⅞	5	4	⅝	4½	1	2¼	2	2	¼	134	155
36"	38½	⅝	5½	26¼	22	1	1¼	8	⅞	5	4	⅝	4½	1	2¼	2	2	¼	154	175
48"	51¼	⅝	5½	33½	29¼	1	1¼	8	⅞	5	4	⅝	4½	1	2¼	2	2	¼	192	213

FIG. 1

anchored by a hub, movement of the bands by sliding is prevented and one can rely on the rods taking up the unbalanced pressure up to the full strength of the rod.

It has also been suggested that the band might be tapped and the pipe drilled so that stud bolts would extend into the cast iron far

enough to give the necessary resistance to the movement of the band by the shearing strength of the bolts plus the frictional resistance of

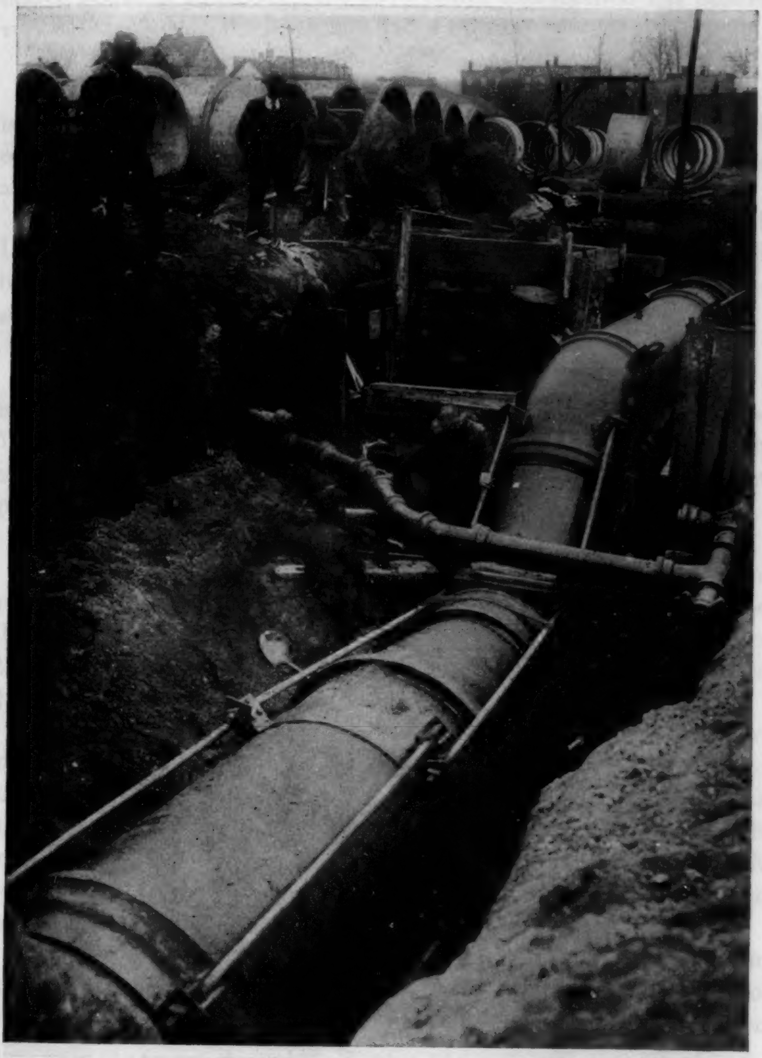


FIG. 2

the band. There are a number of other methods that could be followed in securing the band against movement. The interesting

fact is that dependence on bands that have been clamped tightly around the pipe by the use of bolts is not sufficient when large un-

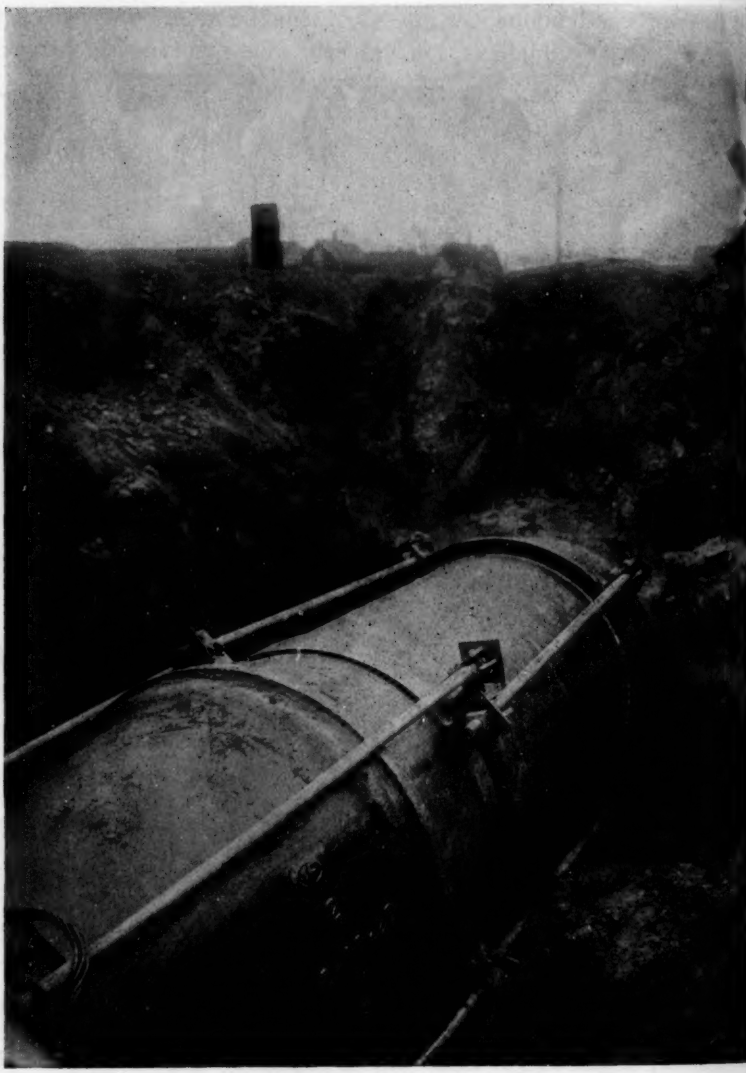


FIG. 3

balanced pressures are to be controlled and from our experience additional resistance to movement of the band is essential.

DISCUSSION

ALEXANDER LINDSAY:[2] It has been our practice to set all 90-degree bends in concrete. We have found this method has proved very satisfactory. We have not had a failure.

MR. BRUSH:[1] We have been rather disinclined to use concrete because we do not know how long it will be before a subway will come along and leave our concrete hanging up in the air with nothing as a backing. It would not be effective under those conditions.

Of course, each community has its own problems to work out. I should think that, in cases where you did not expect your earth backing to be disturbed, the concrete method represented the simplest and most effective method of holding your casting.

L. MURRAY GRANT:[3] I would like to ask Mr. Brush if they are making standard practice now to have lugs put on their specials. We are doing that in Seattle. Of course, we have the problem of shackling on fittings that were put in before we made that standard practice.

MR. BRUSH:[1] Our specials are lugged. You will notice in the pictures that they are lugged. But our difficulty is in our last pipe. There, as I say, it did not seem to us, based on our consideration of the problem, that it would pay to have lugged pipe available when the time came to use it.

What we did here was simply to split up our band. Rather, we made two bands, and just put one band back of the other. We carried one band back to a third band back of the next bell. This was fastened with bolts to the band. Then the pipe would give you the resistance of your stud, plus frictional resistance. Or, putting the stud bolts in the front, it would give you the same thing. There are a number of different methods that could be followed.

This is a very simple method from the viewpoint of the construction engineer. It is not always the simplest, but is merely a method of doing it quickly. Using these bands and bolts and keeping them in stock, you can get and put them on quickly when they are needed.

There is one objection to rods. Some day they will rust out, but that will not occur for many years. We have the rods, but we have

[2] Superintendent, Water Division, Spokane, Wash.
[3] Superintendent, Water Department, Seattle, Wash

not corrosive soil. After about twenty years the rods are nearly as good as when we put them in. I am not at all certain, however, that the time will not come when we will be able to use an amalgam in the joints to hold pipes together and have an extra groove cut possibly on the spigot as well as on the bell end. But for the present we are using rods.

J. E. Gibson:[4] In Charleston we have no very serious trouble. Our pressure is about 40 pounds, but it is in level territory. We have no severe hills. We have to go back about 30 miles to get 100 feet above tide.

We have used bands and rods and concrete and rods in bands. We have used the concrete at the bottom of the cut where we had to cross a railroad track or something of that description, with the rods at the top. We have also used an amalgam of lead, tin and zinc.

In the particular case in which we used that we had an 8- or 12-inch reducer blow out a 24-inch tee. It put the whole plant out of commission. The quickest thing to do was to push that reducer back into place and run in the amalgam. We caulked it up. We had to be rather careful in caulking so as not to split the bell because the amalgam was quite hard.

After that we had no trouble, although we did not use any band or backing. We were working under 135 pounds pressure. Since then we have cut the pressure and the maximum is about 90 at that particular point.

With us on the sea coast we find it objectionable to use bands and rods unless we encase them in concrete because the soil salt content seems to destroy the iron very rapidly. Wherever we use the bands, if it is going to be a permanent job, we encase it in a thin layer of concrete. Of course, we have no subway work that would interfere.

Chairman MacDonald: I would like to ask Mr. Brush if it is the policy of his department in making extensions of 6- and 8-inch cast iron services to enter buildings at right angles to the building. Is there any possibility of a band being installed immediately inside?

Mr. Brush:[1] The connections to the building are installed by the owner and at his expense. We, however, inspect such connections.

[4] Manager and Engineer, Water Department, Charleston, S. C.

There is usually no difficulty with 6-inch pipe with the pressures that we have, which are as high as 150 pounds, but generally are around 40 or 50 pounds. I assume you refer to the question of blowing out?

MR. MACDONALD: Yes.

MR. BRUSH:[1] I do not recall any particular difficulty we have had with blowing out. We have used an amalgam of tin and lead very successfully on joints in the high pressure system. We had a high pressure job on Coney Island where we operate at about 150 pounds and test up to 250 pounds, without any other support than the shearing strength of the amalgam joint. There we have a double groove on the spigot side of the joint and the bell side of the joint.

That joint would not hold if there were to be simply a slight irregularity in the surface of the pipe on the spigot end. The groove should be cut in it. When we get up into the 12-inch or larger pipe we generally find that is when our pipe moves. In smaller pipes that handle around 50 pounds pressure we do not tie the pipe together. Our high pressure in the main part of the city where we use 350 pounds has worked out very satisfactorily with our double groove joint. That pipe has been tested to 450 pounds per square inch without any difficulty.

Does that answer the question?

CHAIRMAN MACDONALD: Are those large 12-inch connections banded?

MR. BRUSH: They are not banded. We use the amalgam. When we use the amalgam we depend on that for strength.

W. J. ORUM:[5] Have you experienced any difficulty with electrolysis?

MR. BRUSH: Electrolysis has very little effect. In fact, the only serious electrolysis we had was about a year ago when the city itself was damaging a new steel line. We were trying the experiment of running a trolley car system down in Staten Island and we took a broken down system and were trying to operate it on a five-cent fare.

[5] Commissioner, Water Works, Montgomery, Ala.

It did not work out very well. We paid the bill for the damage the current did to the steel pipe.

At present we are about to spend from $10,000 to $12,000 to have 800 feet of that pipe uncovered and welded. The pipe line now is about 4 years old. With the exception of that one instance, where it was the city's own fault, we have had slight trouble with electrolysis.

Chairman MacDonald: With reference to these high pressure connections for fire, of 6, 8, 10 and 12 inches; in the city of Ottawa one good method has been to design the casting which projects through the wall with a large collar on it. That collar is faced up against the outside wall of the concrete and embedded in the concrete also. Have you used that method at all?

Mr. Brush: We have not used the system Mr. McDonald refers to as the city does not put in the connection to the building. I believe that the most effective joint that could be used at or near the outside face of the building would be a ball and socket joint, such as is used in submerged pipe lines. If the bell of such a joint were just outside of the building, a settlement either of the building or of the pipe could be taken care of by the movement of the pipe in the bell without causing a break in the pipe.

MAPS AND FIELD RECORDS[1]

By Edgar K. Wilson[2]

The purpose of this paper is to present methods of keeping adequate maps and field records for water works plants in forms available for use outside the office and of keeping these records closely up to date.

This matter may seem elementary to many water works operators and to all such I wish to extend my hearty congratulations, for they are the fortunate ones to whom a valuable legacy of accurate records has been handed down, and who have had the foresight and wisdom to keep these records faithfully throughout their administrations, in spite of the difficulties of lack of help and time, which is the lot of most of our members. They are the fortunate ones who can refer to their maps and tell at a glance whether the connection at a certain intersection is by means of two tees or a cross, and whether there is a sleeve in the middle of a block.

However, during many years experience in water waste surveys as a pitometer engineer, and after examining the records of many water works plants, large and small, privately and publicly owned, it has become evident that the fortunate ones mentioned are not in the majority.

Let us see what The National Board of Fire Underwriters has to say on this subject in their reports.

New Jersey: "Large scale sectional plans . . . . in many cases do not indicate whether or not mains are connected at crossings. In the past much dependence has been placed on the memory of employees, but efforts are now being made to build up accurate records."

Texas: "Records of recent construction are available but of the older works are not in convenient form and are incomplete."

Texas: "Most of the older work is not well mapped."

Ohio: "Records of locations of important gate valves are on file, but not in convenient form for outside use and dependence is largely placed on the knowledge of older employees for detail locations."

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[1] Presented before the New York Section meeting, April 25, 1930.

[2] Chief Engineer, The Pitometer Company, New York, N. Y.

These citations are from reports on cities of considerable size, and might be multiplied, if necessary; but the few mentioned will indicate that the Underwriters do not find conditions entirely ideal.

From recent reports of water waste surveys made by the Pitometer Company the following extracts are taken:

New Jersey: "Valve records are very incomplete, totally unindexed, and poorly kept."

New York: "The engineering records and data from the installation of the present system . . . . up to recent years are inadequate."

New Jersey: Recommended "that records be made and kept of the location of taps and service boxes."

Illinois: Recommended "that the map of the distribution system be corrected and brought up to date."

Michigan: Recommended "that a suitable map be made of the entire system" and "be kept up to date."

Texas: "It was found that the existing maps of the system were inaccurate in many respects and locations of valves in most cases had not been kept."

These extracts are from reports on cities of sufficient size to give considerable importance to the lack of adequate records; and this list could also be multiplied if necessary. Enough cases have been cited of representative cities to indicate that this paper may be of some value in bringing about a more desirable condition.

Yet it is not surprising that a lack of records exists, especially in the older construction, when one considers the method of growth of most of the plants. It has rarely happened that a town has existed without a water works system until it was of considerable size, and has then constructed a plant complete. The usual history has been that of the small plant, perhaps using a spring fed supply and furnishing water to a limited number of people. In those early days a few valves controlled the system. In the average small community it is safe to say that a large percentage of the entire male population old enough to walk knew exactly where every pipe was laid and where every valve was set. At all events, the man in charge of the water system had this knowledge, and he added to the store in his head as extensions were made. After a while, as the system grew and more consumers were added, the superintendent, or whatever title he was given, added to his staff; and usually among these newcomers one or more men would gradually acquire knowledge which had formerly been stored and preserved only in the memory of the superintendent; and the memories of such men were now depended upon for all cases

where connections or repairs were to be made. This method is still in operation even in some cases where the systems are of considerable size.

The water works plant ages and so do the men. Employees pass on or perhaps their memories are not so good as they have been; and their store of knowledge is lost. In one case one of our large cities depended practically entirely on the memories of two men for connection and valve locations. As they grew too old they were pensioned and laid off; but almost immediately they were recalled because nobody else knew what to do and no adequate maps or records were available. At once the city began work on a very complete set of maps, sending these men out at every available opportunity with others qualified to make the measurements and sketches of connections for permanent records. In this instance the old employees were glad to give up their information; and finally the records were in such shape that no further difficulty from this source was encountered, and the two veterans could take a well earned rest.

There is another side to the picture. Often the old employees have an idea that all their knowledge is their own personal property, even those data which are recorded in the office files; and it has sometimes happened that a disgrunted superintendent, superseded or resigned from his office, has taken with him or destroyed records essential to the operation of the plant, causing endless confusion and expense before the records could be recovered or replaced.

One city had its system divided into four parts, each under the control of one man who was thoroughly familiar with the valve system and connections, and who was responsible for all shutting down of mains in his section. Intense jealousy existed between these men and the rest of the employees in the department. The feeling seemed to be that, if they could prevent the other employees from acquiring any knowledge of the valve system their own jobs would be forever assured. Just what would happen if they died or were incapacitated seemed not to enter into their consideration at all. This unfortunate condition was materially assisted by the fact that accurate maps and records were not available and the memory was especially important. In this case the four section foremen were unwilling to impart any information; and when forced to do so, in some instances gave false data; so that everything they said had to be checked in the field before permanent records could be made. In spite of this opposition, however, this city now has very complete sets of records.

The foregoing remarks are not presented as an argument for a proposition which needs no argument. Every water works operator, no matter what the size of his plant, would feel much safer and easier in his mind if he knew that he had maps and records instantly available for use in emergency.

NATURE OF MAPS DESIRABLE

Maps are of various sizes, scales, and degree of detail, according to what their use is to be. First, might be mentioned a general map covering the entire system without much detail. If the source of supply is at some distance from the point of consumption, in order to get the information on a sheet of reasonable size the scale may need to be very much reduced, and is often made small enough so that the map will go on a letter sized sheet. This would obviously be too small for much detail, and would be used principally for reference to show the important points of the system. Details along the line, such as pumping stations, gate houses, dams, reservoirs, or stand pipes, would then have individual plans of appropriate scale for their construction, while the distribution system would, of course, have its own larger scale map.

Next in order would be the detail plans and maps as mentioned above, and where large conduit lines are laid requiring maps and profiles these records should also be filed for future reference. In many such cases the exact location of every joint is measured and recorded, as well as the location of all specials in the conduit. All such maps will have scales sufficiently large to give the required information.

After the distribution system map would come the valve location maps for field use if this system is used for recording their location. These will be on a still larger scale to allow the measurements to be clearly shown.

An intermediate set of sectional maps may be desirable between the distribution system map and the valve location sketches.

Thus we have for a complete set: (1) A general map showing the entire system in a diagrammatic form on small scale; (2) detail maps and plans of special structures and of the conduit line; (3) complete map of the distribution system; (4) sectional maps of the distribution system; and (5) large scale intersection maps showing the valve locations with measurements.

The number and kinds of maps which may be desirable for a given

system will vary with the size of the system and also with the class of data which it is desired to present. One map if sufficiently large scaled may be all that is required for a small plant to show the details of the distribution system. On such a map may be shown location measurements of valves, and even of services; but usually other forms of records for these details will be found better, because the map may be larger than convenient.

Many people have the idea that the larger the scale to which a map is drawn and the more space it covers, the more valuable it will be. This is true only to a certain extent. In such a system as just mentioned it may be well to have everything shown on a large scale wall map, but a map of smaller scale which will show the mains, valves and hydrants, will be much easier to handle and in most cases will give the information usually required. When it is considered that a map 3 feet square will cover an area of about 9 or 10 square miles at a scale of 500 feet to the inch, it will be seen that for most small plants a map of this size will be more than ample for a general map. As the system increases it may be necessary to decrease the scale to as small as 1000 feet to the inch to keep the dimensions down to a resonable size; but it is rarely advisable to go below this scale for a map of the distribution system. It would be better to divide the map into two or more sections and use a larger scale to maintain accuracy and legibility.

The maps which will be discussed are those applying principally to the distribution system, which in many cases will also include the sources of supply as well.

Some operators of small plants may be deterred from making maps because they do not know anything about drafting. This feeling should not keep them from making a map, as any map is better than no map at all, as long as it is fairly accurate and complete. Almost the smallest villages have some kind of street map available; in the county atlas, perhaps, if other sources fail. With a copy of this as a foundation it is possible to draw in the mains between the street lines and show with considerable accuracy the locations of the accessories. There you have a map of the system. To get reproductions there is often a student in the village who will be glad for a small sum, to make a tracing from which as many blueprints may be made as desired. Where the plant is of sufficient size to warrant the expense, it may be more satisfactory to have a local surveyor make up a map, or to employ someone in the department capable of obtaining the necessary

field data, making maps and keeping the records up to date. In a fair sized water works in a fast growing community there will usually be plenty of work along these lines to keep at least one man busy most of the time if the work is properly done.

The question of what should be shown on the map is important, and, as indicated previously, depends to a considerable extent on the purposes for which the map is to be used, and the size of the system; since the larger systems may need maps of several different scales to clearly represent the conditions.

In such cases the smallest scale map of the distribution system should show all mains, valves and hydrants, as well as the more important service connections, such as factory supplies. It may not be possible to show much detail on such a map, but the non-connected mains crossing each other should be indicated by a break in the line of one main or a loop of one across the other. These are the usual methods, although in some cities intersections show a blob of ink at the crossing, if the mains are connected, the blob being omitted if not connected. Important intersections too intricate to show in proper detail on the general map, may be circled and an enlarged section transferred to the margin of the map which will give the details.

The larger scale sectional maps will naturally show more detail than the general maps and in some cities the scale of the sectional maps is large enough so that even the location measurements for the services can be shown. This is not usual, although the services themselves may be shown on such a map, without undue confusion. Any information regarding the location of the main, depth of cover, etc., will be of value. It is often wise to indicate also the location of sewers, electric and telephone conduits, gas mains and other street structures.

Coming now to the valve location or intersection maps it is desirable that these be kept to a size convenient for use in outside work. A size should be chosen which will fit some standard loose leaf binder, so that revised sheets may replace those made obsolete from time to time. As a guide to the size needed, with a scale of 20 feet to the inch, a sheet with available area inside margins and binding edge of 4 inches minimum will allow for 80 feet. As the sheet will be rectangular this would give over 100 feet on the long dimension. With this scale it is possible to show considerable detail for the intersections, and still be able to give the measurement to the valves clearly without confusion.

LOCATING VALVES

The method of locating valves depends to a considerable extent on local conditions. Perhaps the most usual way is by the measurement from two permanent points to the valve in a direct line. This is undoubtedly the most accurate method, but requires swinging a tape from one point through an arc and then intersecting the arc by swinging the tape from the second reference point. This is sometimes a difficult job on a busy intersection. Another way of locating the valve is by a measurement along the nearest curb or property line and then a second measurement at right angles out to the valve. Where the main is laid near the curb this method is very good and often the measurements can be recorded in intelligible form without the use of sketches. This is an important matter in a large city, since a book large enough to contain sketches of all the valve locations would be very unwieldy. Where valve boxes are showing on the surface it is very easy to identify them in this way, but it is sometimes not easy for the workmen to come near enough to the right angle to hit the valve if it is very far out in the street. The use of a valve locator or dip needle in connection should make this method superior to the sketches; but the fact that so much other information beside the measurements may be shown on the sketches, may, in the case of the smaller plants at least, make the intersection plats of sufficiently greater value to warrant their production. In this connection a word might be said as to the permanency of reference points. A few years ago during the construction of a large water conduit, an important angle point was located by measurements to a spike in the root of an elm tree 3 feet through, a telephone pole, and a fence post. As the construction work would start in a short time it was thought that these points would be sufficiently permanent, but events proved otherwise. Only two weeks later, when we were ready to stake the line, we found that the fence had been torn down, the telephone pole on the opposite side of the street had been moved, and, hardest blow of all, the stump of the big elm was just disappearing down the street in tow of a steam roller.

Locations should be standardized throughout the system, that is, if measurements are taken from the curb line in some sections the curb line should be used throughout. It is probably better to use the property line for such measurements, since curb lines are more apt to be moved than property lines.

In some cities it is possible to place permanent marks of some kind on the buildings opposite the valve, and to show the distance out from the marker. This is a very good plan where it can be used, but would be of very little value in a town such as I live in where there is a 35-foot setback in the residential sections—and mostly residential sections with no fences. Such a mark makes it simpler to find a valve especially in winter when it is sometimes difficult to find even the curb line. In one city a composition brass casting is used for a marker, and has blanks on which to stamp the distance out and the size of the valve. This would seem preferable to painted markers, if it is possible to keep the brass plates in place. Such markers are to be considered only as supplementary to intersection maps and valve location records. In many places the valves are so located that a marker cannot be placed to indicate their location, and it will often be found that location by two intersecting measurements is the only method which can be successfully employed.

CONVENTIONAL SIGNS AS KEYS

Judgment must be used in laying out the map for drafting. It is useless to say that the top of the map should be north because in many cases it will be found that unless the arrow is shifted the general map will be unwieldy to handle; although since this convention is known to almost everyone it is well to make the smaller maps in this manner.

The method of indicating the various types of construction is important. As to the mains we can do no better than follow the conventional signs used by the National Board of Fire Underwriters, unless in certain systems other conventions have been used for so long a time that it will cause great confusion to change them. In one large city the sectional maps are beautifully drawn with double lines for each main, and show every fitting in its proper location. Where a main crosses another the method of connection is shown by cross or tee, or if not connected it is clearly indicated whether the main crosses under or over. It would seem that such an elaborate set of maps would be unduly expensive and that the same results could be obtained by a much less costly method. Attempts have been made from time to time to evolve conventional signs which may be depended upon to show at a glance the size of main in a rational way. Usually these depend upon a system of dots and dashes, and while they appear to be very efficient in theory, they are usually hard

to follow in practice on a drafting board without materially slowing the work of the draftsman. The method of indicating pipe sizes by different colored lines is also good in theory, but not so good in practice. If we could get good colors sufficiently differentiated so that a glance would show the size of the main, and, above all, which would make blue prints in the same colors, this method would be ideal. The second difficulty is the principal argument against colors, but most of us have seen drawings showing water mains in different colors and have been unable to tell whether the street under observation had a red line meaning 6-inch or a brown line meaning 16-inch, while the blues and greens and yellows which had looked so conspicuous when the map was first drawn had faded to a dingy shade which might be almost anything.

The Underwriters conventions are well known, the different kinds of lines are not complicated, there is no difficulty in blue printing, and while it is necessary to label the larger sizes of pipe, by drawing the different sizes a little heavier as they become larger, it is not difficult to get the size without trouble. Why not take advantage of their widely followed practice?

For clearness it is better to make general maps in skeleton form rather than to use the street lines. This applies, of course, to the smaller scale maps on which it is not necessary to record measurements. Where two or more mains are laid in the same street, by using the skeleton method, each main can be shown sufficiently separated from the others to give opportunity to show any cross connections or other detail which may need to be inserted without danger that they will be overlooked because of the close proximity of the mains.

Valves are sometimes indicated by a circle or cross, but a short line across the main is simpler; and when this symbol is used a circle can be drawn around it to indicate special conditions, such as a closed boundary valve.

Dead ends should be definitely noted by means of a short arc of a circle drawn across the end. This is especially necessary where a main dead ends just before meeting another main, as it is easy to mistake such a point for a slip on the part of the draftsman.

PRESERVING AND FILING MAPS

If you go into almost any water works office in a large city and look over the tracings of the maps, you will find a very expensive set of tracings, with edges damaged, and often so crumpled and with so

much dirt on the drawings that only a very poor print can be made. After a time these conditions become so bad that the portions of the sheet near the edges and corners are illegible and new sheets must be made. It is entirely possible and relatively inexpensive, when the tracings are new, to have reproductions made on tracing cloth which can then be used for office work, while the originals are filed away until other reproductions are needed. A sheet which would cost perhaps a hundred dollars to retrace can be obtained for a very small fraction of that amount and can then be used to record changes and additions, the master tracing being brought up to date once a year. When the working tracing begins to be illegible a new reproduction can be made and the process repeated.

The filing of maps and especially of tracings is of some importance. In a large plant where there are many drawings to take care of it is not hard to get proper files, but in a small water works where there are only a few maps to look after, a regular file for them would ordinarily not be warranted. It is much better to file maps flat, whether tracings or prints. Sometimes this is very inconvenient, and then it is better to fold the blue prints to letter size and file them in a regular letter filing cabinet or in a desk drawer. It is important to keep tracings clean and dry. If they get damp the surface becomes opaque, and dust, if it has a chance will work into them and cause the same result. Probably the most simple method of taking care of a small number of tracings is by storing in tin cans which are made for the purpose. When they are rolled many people believe that it is better to roll them with the drawing side on the outside of the roll, instead of inside as is usually done. By this method when a tracing is unrolled on a table it will lie flat of its own weight, where otherwise the ends will curl up and have to be held down. However they are rolled they should never be rolled tightly. How many times do we see valuable tracings rolled so tightly that it is almost impossible ever to make them lie flat. It should be remembered that anything which makes more handling necessary shortens the life of a tracing.

Tracings of small intersection plats may best be filed in a loose leaf binder similar to those which are used for the blue print copies used in the field. A complete index of these plats is essential.

RECORDS OF SERVICES

Locations of services have been mentioned in connection with some of the large scale maps. While the measurements may be and sometimes are recorded in this manner, such procedure does not give much

of any information beyond the exact location on the ground. In order to keep proper records of the services, material as well as location, each service should have a separate sheet, which will give the date of laying, the street location, the measured locations of the corporation cock, and curb cock, the size of tap, the kind and quantity of material used, as well as any other items which may be pertinent to a specific service. These records are well adapted to card index filing, and may be filed either by streets or by serial number, the first being preferable. If the serial filing is used an index is also necessary to locate services by streets. In some cases a combination of the two is employed, the services being listed serially in a book, and a card index being maintained by streets.

VALVE OPERATION RECORDS

Another system of field records which is not employed to the extent which it should be is that of valve operation records. Such a record should show the size, number of turns, direction of opening or closing, date of installation and make; and should have room for several operation notes, giving the date of closing, opening and reason therefor. In almost all water works plants it is well known that the valves are not operated frequently enough to keep them in good working order. It is advisable to operate every valve at least once every year; but while some valves get fairly regular attention in the usual run of water works operation, many are not touched for many months at a stretch. The use of such a record will make it possible to see that these idle valves are in operating condition ready for the time when they may be badly needed.

There are other parts of the water works operations which need special records, such as meters, house inspections and fixtures; but these are more closely connected with the accounting department than with the outside maintenance and construction work, and discussion of these is not pertinent to this paper.

SUMMARY

Get a map or as many maps as the size of your system may require; keep them up to date; keep accurate valve and service location records; carry such other records as may be necessary for efficient operation. These will take time and money to prepare, especially in some of the larger systems where they have been allowed to be neglected, but one broken main which is not promptly shut down because of lack of proper valve records will perhaps do more damage than would pay the cost of these records several times over.

EFFICIENCY IN METER READING[1]

By L. D. Shively[2]

This is one of the most important, and probably most neglected subjects confronting the water companies today.

In the successful operation of a water company we must have efficient management, efficient engineering, efficient office operation, etc. It is equally important that we have efficient meter reading.

The difference between an efficient and an inefficient meter reader is that one thinks about his work while the other does not. The difference between an efficient and an inefficient meter reading department is measured in thousands of dollars of revenue.

We have read many articles in the engineering journals and Proceedings of the American Water Works Association recording the removal of meters for test, some of which were showing an under registration of from 10 to 30 percent, while others had ceased registering, possibly months before. This surely indicated an inefficient meter reading department. The efficient reader makes a study of the needs of the various types of consumers; is quick to notice any variation in their monthly consumption, and is constantly watching the meters to see that they are registering as nearly 100 percent as is possible. The inefficient reader reads a meter and passes on to the next one, giving little thought to variation in consumption or to the accuracy of the meter. In other words, he is just a "statement taker."

REQUIREMENTS FOR GOOD METER READER

In the building of an efficient meter reading department, there are many things to be considered, the first being the type of men best fitted for this work. They must possess tact, and be able to think clearly and accurately. They must be courteous, have a pleasant disposition, and keep smiling through the most trying circumstances. Although they have to help move kitchen cabinets, tables, beds, rugs, etc., from trap doors before they can enter the basement to read the

[1] Presented before the Indiana Section meeting, March 21, 1930.

[2] Chief Meter Reader, Indianapolis Water Company, Indianapolis, Ind.

meter, they must still say "thank you" when they are through. No employe of the company meets more people than does the meter reader. In fact, he is the only employe of the company some customers ever see. When they get their bill they mail a check, and if they have a complaint they write a letter. They seldom, if ever, contact with the office. It is, therefore, highly important that the reader create the most favorable impression possible on the people with whom he comes in contact. In the minds of the majority of the customers he is the water company. He is its one and only representative.

LOCATION OF METER

The next matter of importance is the location of the meter. If we expect the meter readers to give the best service, we should be very particular where we set the meter. The meter must be read at frequent intervals, therefore, it should not be placed behind a furnace, in a coal bin, or in a cellar that is little more than a hole in the ground. If in a meter box, we should be sure the lid of that box is slightly above ground level. We should not expect a man to keep smiling, or be efficient, if he has to crawl behind furnaces, climb over coal and ashes, or remove 2 or 3 inches of dirt from pit lids.

Another matter of vital importance is the routing of the meters. This must be done so that a reader will not be compelled to take unnecessary steps. Our company is now completing a routing system, and our experience indicates that it measures close to 100 percent efficient. The city was divided, as nearly as possible, into sections five squares each way. Each section was numbered in such a way that if any changes are made such as cutting new streets, the section can be renumbered without affecting any other part of the routing system. Instead of the reader zig-zagging back and forth across the street, weaving in and out among fast traveling automobiles, etc., he goes up one side of the main street and back the other, reading the cross streets for one square by the herring-bone system.

A blue print was made of each section; the streets marked off in 20-foot spaces, and two numbers allotted to each space. Inspectors took the blue prints out in the territory and placed the property numbers on these prints in their exact location. Although the house numbers may be changed the meter account numbers remain the same. We have been more than four years perfecting and completing this system, but our experience proves that the time was

well spent. The increased efficiency of our meter readers far outweighs the expense of instituting the new routing system.

DUTIES OF METER READER

The next question of importance is how many readings should a reader procure in a day. This would be easy to answer, if all we expected of a reader was to get a reading of the meter.

A year or so ago Mr. F. C. Jordan called me into his office and asked me to read an article written by a superintendent of a southern water company, in which the superintendent praised his meter readers, and said they were undoubtedly the best in the country. According to his report the readers averaged 274 readings per day. Our quota is 180. Was the mere fact that their readers procured 274 readings per day an indication that they were efficient? Our experience indicates that a reader cannot read 274 meters per day and be efficient in all matters properly coming under his charge. There are other points to be considered that we feel are just as important as getting a reading of the meter. The trend of our thought today is that these men are water company representatives and not merely meter readers, and our system is being remodeled along that line. At the present time these representatives have six other duties, namely: (1) Packing the stuffing boxes of the meters; (2) making investigations to determine the cause of high consumption; (3) testing the meters to make sure they are registering all the water that passes through them; (4) locating consumers who vacate houses supplied with water by us, without notifying us; (5) making inspections where houses are occupied and meter sheet shows water off; (6) making take-out and test tickets where the meter has registered a certain quantity as designated in our Standard Practice.

The first item "packing stuffing boxes," is of importance to the water company, because any leak at this point is the company's loss. Also, if a leak at this point continues, the small hole in the hood where the water is supposed to escape becomes clogged, and the water cannot get out and corrosion sets in in the clock gears and destroys them in a very short time. We believe that the meter reader should pack the stuffing boxes when they are leaking, as it only takes him from three to five minutes. If this work is done by the distribution department, the service man must make a special trip, making the packing of the meter more expensive, and causing an added annoyance to the consumer.

The second item, "Making leak inspections," is of importance to the customer, and is also important to the company when viewed from the standpoint of public relations. It is our policy, although we do not consider it our duty, to help a consumer find the cause of any high consumption. This service is sometimes not properly appreciated, but we know it is appreciated by some because many have written or called at the office and expressed their thanks.

During the month of December, 1929, our meter reading department made 552 leak inspections, and the monthly average for the year was about 400. If this valuable service is to be rendered the meter reader is the logical man to perform it.

The third item, "testing meters to make sure they are registering all the water that passes through them," is the most important of all to the water company, and applies especially to the commercial and industrial meters. It is necessary that a meter reader be able to read a meter correctly, and know that the meter is registering all of the water that passes through it. Some meter readers are expert "statement takers," but unless they familiarize themselves with the different types of consumers, and learn to know how much water they should use, they cannot be classed as efficient.

The fourth item, "locating consumers who vacate houses supplied by us without notifying us," is also of great importance to the water company, and a meter reader can render invaluable service to the collection department by procuring this information.

The fifth item, "making inspections where houses are occupied and reader's sheet shows water off," is important as there is hardly a month that the readers do not find one or more properties where the water is on without the company's knowledge. A reader should also watch houses that have no city water. Sometimes a plumber installs fixtures in a house; turns the water on, but does not immediately notify the office.

The sixth item, "making take-out and test tickets where meter has registered a certain amount," is one that must be taken care of by the meter reader. Instead of ordering a meter out for test after it has been in service a certain number of years, we now order it out after it has registered a certain quantity of water. This does not mean that all meters will be left in until they have registered a specified amount. *If at any time a reader feels* that a meter is slowing down, and not registering accurately, it is his duty to order it out for testing.

The question of "missed meters" is becoming a more serious prob-

lem every day. Should we let the readers use pass keys to gain admittance to homes? We do not believe they should unless we are granted permission by the consumer. If the readers, as they go around the side of the house, will call out "water man," loud enough to be heard in the house, but not in the next square, it will save them time in gaining admittance, and lessen the number of "missed meters." The occupant knows who is coming, and will not hesitate to let him in. On the other hand, if the reader does not call but just knocks on the door, the occupant frequently does not answer thinking it is some agent or peddler.

We have meters in a number of homes, where both husband and wife work, and there is no one at home during the day. We have been furnished keys to many of these, so that we may have access. These keys are kept in a locked room except when the readers are using them.

Another matter that may seem rather insignificant to some readers, but is really very important, is the marking of meter locations on the readers' sheets. Especially is this true where the meter is back of a furnace, or in some unusual location. If this is done it will speed up the work of the reader, and make it much easier.

UNIFORMS FOR METER READERS

Two years ago our company adopted the policy of uniforming the meter readers. Frequently we are asked why we did this. There are three main reasons. In the first place it makes it easier for them to gain admittance to homes. Second, it gives the reader a neater appearance, provided he takes the proper care of his uniform. Third, it is a protection to both the company and the customer. In this day of "fake meter readers," "telephone inspectors," etc., it is important that we protect our customers as far as possible, and the uniforms serve this purpose. The cost of the uniforms, which is about one-seventh of one cent per meter reading, is far over-balanced by the increased efficiency of our readers.

We have outlined the duties of a meter reader in addition to the reading of the meter. He should be given proper credit for this additional work. We should not expect the man who packs 10 or 12 meters, and makes a like number of inspections, to read as many meters in a day, as the man who does not pack any meters or make any inspections.

RATING AND TRAINING METER READERS

A logical plan is to use a point system. Instead of so many readings per day, we should require a man to make so many points. Allow one point for each *correct* meter reading, one-half point for each missed meter, two points for packing a meter, two points for making a leak inspection, and one point for locating a consumer who has moved without notifying us.

When a man is employed as a meter reader by the Indianapolis Water Company, he is first taught how to read meters, and then is sent to our meter repair department for two or three weeks training. Here he learns the construction and operation of all types and sizes of meters. We find this schooling invaluable. Also, in order to develop the meter readers to the degree of efficiency they should attain, and make them real representatives of the Company, the Indianapolis Water Company has adopted the policy of holding regular meetings of the Commercial Department employes, including bookkeepers, billers, contract window men, telephone operators, and meter readers. At these meetings the readers are brought face to face with the problems confronting the office employes. They learn how much trouble may be caused by their mistakes of omission or commission.

We must constantly impress upon their minds that their position is one of the most important in the Water Company Organization. They should not be just ordinary meter readers, capable only of transferring a reading from a meter dial to a reader's sheet, but they should be intelligent representatives of the Water Company in the homes and business places that they visit.

WATER SOFTENING PRACTICE[1]

By Charles P. Hoover[2]

Although the art of water softening, with lime, dates back for more than 100 years, its development in municipal water softening plants has come during the last few years.

The developments in water softening practices, during this recent period, have met one of the great needs of humanity, for it is necessary to have soft water to keep clean and to operate successfully many industrial plants. There are a number of communities, both large and small, where almost every family is engaged in water softening, when the community through a municipal plant, could produce soft water at a much less cost at a central source of supply rather than in a thousand homes. Water softening now has an economic background. It has been demonstrated that it is cheaper to pay a reasonable charge for central softening than to continue the use of a hard supply.

In those communities where central softening plants have not been installed in order to meet the problem of hard water, many industrial plants have built their own softeners and in the home, cisterns, individual softeners, excess soap and so called laundry aids such as borax, sal-soda and tri-sodium phosphate are used. The most universally used procedure is to neutralize hardness, or soften with excess soap.

One pound of lime costing ½ cent will soften as much water as 20 pounds of soap costing 2 or 3 dollars. Therefore, the consumer who softens with soap seldom realizes that in order to meet his problem of hard water he has adopted not only the most unsatisfactory, but also the most expensive procedure.

For years the advantages of lime and soda-ash softening were offset to a certain extent by disadvantages. Difficulties arose in operating sand filters on account of incrustation of the sand with calcium carbonate. Water mains, service pipes and meters became clogged with heavy deposits. When water contained an excess of causticity it was unpalatable. Softening of surface supplies during flood

[1] Presented before the Missouri Valley Section meeting, November 6, 1930.

[2] Chemist in Charge, Water Softening and Purification Works, Columbus, O.

periods was unsatisfactory and limited results only could be obtained in the reduction of hardness.

OVERCOMING LIMITATIONS OF LIME SODA-ASH WATER SOFTENING BY KNOWN METHODS

Hot process. The hot process softener is similar to the cold softener, except that it is usually built in conjunction with an open feed water heater and the reaction period is less because heat accelerates the chemical reactions.

Excess lime and soda-ash treatment. This consists in overtreating the water with lime. Usually 2 or 3 grains excess lime per gallon are sufficient to precipitate the magnesium almost completely. Then neutralize the excess lime with soda-ash, converting all the causticity to sodium causticity. The soda-ash required is that necessary to combine with the non-carbonate hardness of the water and excess lime.

The hot process and excess treatment are now used almost exclusively for softening boiler feed water.

Split treatment. At older softening plants, not equipped to use modern practice, it is advantageous to use split treatment; that is, over-treating as large a portion of the hard water as possible to get maximum reduction of hardness and then neutralizing the excess with raw water.

Split treatment reduces the hardness of a given quantity of raw water with a given quantity of chemicals, more than the same quantity of chemicals will, when added to the total supply.

Addition of aluminum compounds. The addition of aluminum compounds, it is believed, converts soluble magnesium salts to insoluble magnesium aluminates or coagulates colloidal precipitates of calcium or magnesium compounds, thus making it possible to remove them by settling or filtration.

RECENT METHODS OVERCOME DISADVANTAGES AND LIMITATIONS

Carbonation. Modern municipal water softening really began in 1921, with the building of the first successful carbonation plant at Defiance, Ohio.

The original purpose of carbonating lime softened water was to stabilize it so as to prevent the formation of deposits in distribution systems, and to keep the sand in sand filters from becoming covered with a coating of calcium carbonate which causes the grains to cement themselves together forming hard lumps.

As originally practiced carbonation was continued to the point where the water had a neutral reaction to phenolphthalein and the hardness of the softened water was increased somewhat because calcium carbonate present in a state of fine division or in colloidal solution was dissolved and remained in solution, whereas without carbonation it was partially removed by filtration.

It was soon observed that, if more lime was used in the softening process and the excess lime or caustic alkalinity neutralized with CO_2, lower hardness could be obtained than had previously been possible at municipal plants. This was noticeable especially in cold water. There was, however, a disadvantage in this procedure, for, in order to get good softening results, it was necessary to carbonate only partially and the water so carbonated was not stable. It was found that sand filters and distribution systems could not be protected from incrustation by partial carbonation.

Excess lime and partial carbonation. The next advance step in softening came when Mr. James Montgomery at Piqua, Ohio, Water Softening Plant, observed that when a substantial excess of lime was used (30 to 40 parts caustic alkalinity), and the softened settled water was carbonated to the point where two times the alkalinity to phenolphthalein when subtracted from the total alkalinity equalled 5, the alkalinity of the softened water, on being filtered through sand filters, always dropped to almost the theoretical solubility limit, that is around 18 or 20 p.p.m. An average of 45 to 60 p.p.m. had previously been considered as being as good as could be expected.

This treatment produces water that does not deposit scale in the distribution system, but it is not so good for the filters. They become coated and hard lumps form as when uncarbonated softened water is filtered through them. A still further advance occurred when it was observed to be an advantage to provide a reaction time period between carbonation and filtration. At Marion, Ohio, where long retention time periods are provided, it was found that by overtreating with lime, settling, then carbonating so as to leave 5 p.p.m. bi-carbonates and again settling for a long period (almost twenty-four hours are provided at Marion), softened water with an alkalinity at the solubility limit of calcium carbonate could be produced without the use of filters. The carbonates crystallize and precipitate into flocks sufficiently large, if time enough is provided, so that they may be removed from the water by plain settling.

Water with too low alkalinity may be produced. During a part of the

year of 1929, low alkalinities were produced at the Columbus, Ohio, plant by excess lime, partial carbonation and filtration. It was soon discovered, however, that it was possible to produce water with too low an alkalinity or carbonate hardness because red water troubles developed in many domestic hot water heating systems.

About this time Mr. Baylis made known his test for determining calcium carbonate solubility equilibrium. Mr. Baylis discovered that, by adding calcium carbonate to water, stirring, settling, filtering, and then titrating for alkalinity, the calcium carbonate solubility equilibrium could be determined. If the addition of calcium carbonate increases the alkalinity, the water is undersaturated with calcium carbonate. If the alkalinity is not changed the water is exactly saturated, whereas, on the other hand, if the alkalinity decreases the water is supersaturated.

Corrosion of pipes and heating systems is prevented in communities, where it otherwise would occur, by an adherent film or coating of calcium carbonate formed from water saturated or supersaturated with calcium carbonate. If water is undersaturated this coating is dissolved and corrosion will occur. If water is supersaturated too much coating is formed and pipe lines become clogged.

It was found in Columbus that the alkalinity or carbonate hardness should be carried so that it would be saturated at 180°F. instead of at ordinary temperatures. A temperature of 180°F. is almost as hot as water should be heated for domestic purposes. Since adopting this schedule, complaints from red water in heating systems have practically ceased.

In making these tests for stability on samples of water that were only partly carbonated it was observed that when calcium carbonate was added to water that had been treated with excess lime and carbonated with just enough CO_2 to neutralize the causticity, remarkably low alkalinities could be produced. For instance, the alkalinity of carbonated water from the settling basins at Columbus was reduced from 68 to 17 p.p.m. A sample of ice cold river water having an alkalinity of 180 p.p.m. was treated with lime and stirred for five minutes. The analysis following the five minutes' stirring was as follows:

Total alkalinity	220
Alkalinity to phenolphthalein	135
Caustic alkalinity	50

Enough CO_2 was added to neutralize the caustic alkalinity and calcium carbonate was added with the result that the total alkalinity was reduced to a 18 p.p.m. The total time for completion of the experiment was fifteen minutes. By old processes, equivalent results could probably not be obtained even with reaction periods of many hours or even days.

EXCESS LIME, PARTIAL RECARBONATION AND RETURN SLUDGE

The results of these experiments suggested still another new, and we believe, a very valuable development by which the carbonate hardness of water by lime softening can be reduced to the theoretical limit. The use of this process it is believed will overcome filter sand problems as well as distribution troubles, because the normal carbonates are removed ahead of the filters.

Instruction for using process

1. Use from 25 to 50 p.p.m. of lime in excess of that theoretically required to combine with the free and half-bound carbon dioxide and to precipitate the magnesium; mix.
2. Settle out settleable precipitates.
3. Neutralize excess lime with carbon-dioxide gas ($Ca(OH)_2 + CO_2 = CaCO_3$).
4. Mix with sludge produced from water softening reactions.
5. Settle.
6. Carbonate if desired.
7. Filter.

For example: A sample of Scioto river water having a carbonate hardness of 144 p.p.m. was mixed with 11 grains of quick lime and settled. The supernatant water showed the following results of analysis in parts per million.

Alkalinity to methyl orange	78
Alkalinity to phenolphthalein	61
Excess lime or caustic alkalinity	44

The sample was carbonated with carbon-dioxide gas until it analyzed as follows:

Alkalinity to methyl orange	60
Alkalinity to phenolphthalein	31
Excess lime or caustic alkalinitv	2

Calcium carbonate was added, stirred for less than one-half minute, allowed to settle and was filtered through filter paper and analyzed with the following results:

Alkalinity to methyl orange	15
Alkalinity to phenolphthalein	4
Excess lime or caustic alkalinity	0

An opportunity for trying out this process on a plant scale has now been provided. The Cedar Rapids, Iowa, plant was designed to use this process and is the first application of it. Until a clarifier mechanism is installed in the second basin the process will, however, probably not be used in its entirety. Several other plants now under construction will soon be ready for operation.

The present operation at Cedar Rapids is now the nearest approach to this process. The water and lime (enough to give about 30 p.p.m. causticity) are first mixed, settled, and then carbonated with enough CO_2 to neutralize the excess lime. It is then mixed with alum, settled and carbonated the second time just ahead of the filters. Since this treatment was started the alkalinity of the filtered water has averaged 35 p.p.m. and the hardness 62; whereas during the first few weeks of operation with the plant operating in accordance with old practice the alkalinities were reduced to about 70 to 72 and the hardness to about 97 p.p.m.

Process well adopted to softening turbid surface water. Some softening plants handling surface water are required to purify and remove turbidity as well as to soften. In times of flood the water is very turbid and comparatively soft.

A plant designed as the Cedar Rapids plant lends itself readily to the successful treatment of flood water.

In the usual process of water softening, the lime, soda-ash and alum are applied to the water at practically one point, it being preferable to add the alum either before or after the lime and soda-ash. When treating flood waters the process should be carried on as follows: (1) Mix alum with water; (2) settle out settleable precipitates; (3) mix lime and soda-ash with water; (4) settle; (5) carbonate; (6) filter. It will be noticed that the only difference in this treatment is that alum is added first and preliminary carbonation is omitted. When a flood comes the operator has only to change the point of application of the chemicals and omit preliminary carbonation.

Flood water is usually soft and it would seem sensible to eliminate

the lime, but this cannot be done without greatly increasing the hardness. The large quantity of alum required to effect good coagulation (amounting to from 8 to 10 grains per gallon in many cases) liberates large quantities of CO_2 (amounting to from 40 to 60 p.p.m.). This CO_2 dissolves the precipitated carbonates in the settling basins or on the filter sand and, as a result, the alkalinities may increase from 100 to 140 p.p.m. Theoretically, only enough lime to react with the alum should be necessary, but unfortunately the addition of this quantity results in either very poor or no coagulation. If lime is to be successfully used, in the treatment of low alkalinity flood water, as found in most of our streams, enough must be used to produce caustic alkalinity. Used in this way as much as 10 or 12 grains have been required and the alkalinity of the treated water has in many instances, been twice that of the initial alkalinity. In other words, the lime acts as a hardening reagent rather than a softener. Another objection to the excess lime treatment of flood water is that alumina is partially soluble in waters with a high pH value and as lime increases the pH, the water is apt to contain considerable residual alumina.

LIME-ZEOLITE PRACTICE

Several water softening plants have been built for using a combination of the lime and zeolite processes. In operating a lime-zeolite softening plant, the bicarbonate hardness is precipitated by lime. After settling, carbonating and filtering, the water flows through zeolite softeners to remove the sulphates, chlorides and nitrates. If the hardness is to be reduced to zero, all the water must be passed through the zeolite softeners. If it is desirable to leave some residual hardness in the water, then only a portion of it is so treated. This process is economical when the water being softened contains a large amount of non-carbonate hardness.

A new plant now being built at Findlay, Ohio is designed to use excess lime, settling, partial carbonation, return of sludge, secondary mixing, secondary sedimentation and secondary carbonation and the non-carbonate hardness will be removed by zeolite.

SOFTENING AND IRON REMOVAL BY ZEOLITES

It has recently been demonstrated, in one plant at least, that downflow greensand zeolite softening will remove both ferric and ferrous iron from the water. The zeolite removes iron after its capacity for removing hardness has become exhausted. As a result of this reaction,

an especially promising field for zeolite softening will, in the future, exist in villages or small municipalities because, by its use, water may be pumped directly from the walls through the softener to the distribution system, thus eliminating double pumping, aeration and sand filtration. Hard iron-free or of low iron content water should be mixed with the zero water from the softeners in the proper proportion to produce a finished product of the desired degree of hardness.

Unless the softened water is entirely free from oxygen it is believed that it must be further treated to prevent corrosion in the distribution system. It should contain from 25 to 30 p.p.m. of calcium carbonate and enough soda-ash should be added to increase the pH value to 8.2.

CHEMISTRY OF WATER SOFTENING UNDERSTOOD

The chemistry of water softening is now well understood and an exact prediction of the results to be obtained can be made. Difficulties encountered on account of incrustation and excess causticity have been eliminated, flood water can now be satisfactorily clarified and softened, and water of almost any degree of hardness can now be produced.

In order to soften water meeting these requirements, it is necessary that the plant be specially designed and equipped so that the advantages of modern practices may be utilized.

The improvements in design and equipment all tend toward simplifying operation. Reaction time periods have also been reduced thus making it possible to build smaller basins and lower construction costs.

WATER WASTE CONTROL IN BUFFALO[1]

By Leonard S. Spire[2]

For a great many years the city of Buffalo pumped more water per day for each resident in the city than any other city in the United States. The per capita consumption was enormous and far exceeded any reasonable requirements of domestic and commercial uses. Additional large mains and pumping machinery of unusual capacity had to be installed in order to supply the necessary amount of water. The contributing factors to this large consumption, the low water rates and the sentiment which prevails that with ample supply available not even a reasonable limit upon the consumption could be tolerated, had resulted in futile attempts to permanently curtail this waste.

The report of a survey of Buffalo by the National Board of Fire Underwriters in October, 1909, states "the per capita consumption is enormous, due to the small number of meters, to low water rates and to the futility of attempting to stop leaks and waste by inspection. It is to be regretted that the off-repeated advice of the Bureau Head in regard to setting meters has not been followed since the high domestic consumption reduces the margin available for fire purposes in the street mains and necessitates the installation of pumping machinery of unusual capacity."

Under the heading "Main arteries"—the report states "if the consumption were reduced to a reasonable figure, arteries would be ample in size, and as they are well arranged and connected, would be able to furnish the requisite flows." Again, under the head of "Minor Distributors" the report states "Considering the great length of many of the blocks, the character of the districts and the enormous consumption, 6-inch pipes are inadequate to furnish the service required."

When this survey was made (fiscal year 1909–10), the total consumption for the year was 48,477,000,000 gallons, an average daily

[1] Presented before the New York Section meeting, April 25, 1930.

[2] Chief Pitometer Operator, Division of Water, Buffalo, N. Y.

consumption of 132,759,000 gallons and a daily per capita consumption of 313 gallons, the population being 423,000.

In September, 1916, the National Board of Fire Underwriters made another survey of the city and their report on water consumption was almost identical with that of 1909. Under the heading of "Consumption," it states "The per capita consumption is enormous and far exceeds any reasonable requirements of domestic and commercial uses. Both the per capita and the total consumption have increased in recent years, requiring additional large mains and pumping machinery of unusual capacity. . . . . This can be overcome only by a campaign of education against unnecessary waste, the beginning of which cannot be made too soon."

Under the heading of "Mains" in this report, it says "The carrying capacity of the arterial system should be ample, but owing to the enormous consumption is severely taxed and large friction losses occur, especially in the High Service."

During the fiscal year 1916–17 (when this survey was made) the combined pumpage of water of the two pumping stations was 61,491,000,000 gallons, average daily consumption 168,468,000 gallons, daily per capita consumption 350 gallons, the population being 480,000.

PITOMETER SURVEY

Acting upon the recommendation of the 1916 Survey of the National Board of Fire Underwriters, Commissioner of Public Works, Arthur W. Kreinheder and Water Commissioner, George C. Andrews, decided that the only way to overcome the enormous waste of water would be to have a scientific water waste survey made and institute a campaign of education against unnecessary waste of water. This was commenced in July, 1917, when the City of Buffalo contracted with the Pitometer Company, Engineers, of New York City for a complete survey of the entire city.

This company completed its survey on June 30, 1919. During this two-year period their efforts showed a decrease in consumption of 12,804,000,000 gallons, an average daily consumption decrease of 35,088,000 gallons and a daily per capita decrease of 73 gallons.

On July 1, 1919 a newly formed Pitometer Department was inaugurated as a division of the Bureau of Water and the system used in the survey made by the Pitometer Company was generally carried out. The Pitometer Department continued to "fine comb" the city

for underground leaks and a corps of experienced inspectors made continual inspections of the residences to locate all waste of water through leaking fixtures. As a result of these intensive inspections and the expert work of the Pitometer operators locating all underground leakage by means of the Pitometer, the pumpage of water has steadily decreased year by year.

During the year 1926, the National Board of Fire Underwriters made a survey of the city and under the heading "Consumption" stated "Statistics are based upon the pumpage as recorded by Venturi Meters. As there are practically no private sources, the Municipal Works furnish the entire supply for the city and it is estimated that the industrial uses amount to a daily per capita of 75 gallons. At the time of the National Board Survey in 1916 the per capita consumption averaged 350 gallons per day and was increasing. In 1926 it was 210. This control of consumption and curtailment of waste has been accomplished mainly by the maintenance of a Pitometer Division which operates continuously, measuring the consumption by districts and making inspections for leakage in those districts in which unwarranted increases in consumption occur."

Under the heading "Improvements," the National Board report states "Since the report of 1916 . . . . the percentage of metered services has been increased and a Pitometer Division so capably maintained that a marked decrease in consumption has been affected, which increases the reserve capacity of the distribution system, pumps and boilers, and improves the hydrant pressures generally throughout the City." In their "Conclusions" in this report, under the heading "Consumption" they state "The per capita consumption, which was far in excess of reasonable requirements during the 1916 inspection, has been considerably reduced by the persistent efforts of a Pitometer Division which was established for that purpose. The good work of this Division must necessarily be continued lest the former wasteful ways return and as the present per capita rate is high. Some further reduction may be accomplished if the work of the division is supplemented by a more general use of meters." This report states in its "Recommendations on Consumption" "That systematic effort to control consumption be continued and that a policy of metering all services be adopted and that a sufficient number of meters be installed each year to complete the work in 5 years."

The combined pumpage of both stations for the fiscal year 1928–29 was 43,669,000,000 gallons, average daily consumption 119,642,000

gallons, daily per capita consumption 200 gallons. The total decrease in pumpage from the fiscal year 1916–17 to 1928–29 inclusive was 17,822,000,000 gallons, decrease in average daily consumption 48,826,000 gallons, daily per capita consumption decrease 150 gallons.

The direct benefit from the work of the Pitometer Department has been a reduction of coal, oil and labor at the Pumping Stations, amounting to approximately $90,000.00 a year. The indirect benefits, such as postponing the time when larger feeder mains and additional pumping units will be needed have amounted to several hundred thousand dollars in capital investments.

Two important factors which cause a high rate of pumpage, are promiscuous lawn sprinkling during the summer months and excessive waste of water by those who allow the water to run to prevent freezing during the extreme cold weather in the winter months. These faults can only be entirely remedied by universal metering and it is to be hoped that the "City Fathers" will some day create an ordinance whereby a meter must be installed in each residence. The metered services at present number about 16 percent. Each year Pitometer inspectors report between 25,000 and 30,000 leaking fixtures and between 1500 and 2000 underground service leaks. Careful computation shows that the average amount of water wasted is 1000 gallons per leak per day, or a total of all service and fixture leaks reported during a year of between 25,000,000 and 30,000,000 gallons a day. If the property holders would use more care and repair the leaking fixtures as soon as they are discovered, the Water Bureau would save thousands of dollars yearly in its coal bill. The property holder who keeps his plumbing fixtures repaired has to help pay for the water wasted by his neighbor who is careless and neglects to have the leaking fixtures repaired.

During the winter months more water is wasted by the people in allowing the water to run to prevent freezing than any other season of the year.

In conclusion, I wish to state that Commissioner of Public Works, George F. Fisk and Water Director, Carl L. Lund, have placed their stamp of approval on the activities of the Pitometer Department and have given unstinted support in its deliberations. With this encouragement from the department heads, intense interest is taken in the various problems which confront us and the various crews work in complete harmony so that our results are becoming more and more satisfactory each day.

ADDITIONAL FILTER UNITS FOR THE LEXINGTON WATER COMPANY[1]

By Hugh R. Cramer[2] and A. T. Clark[3]

Early in the seventeen seventies Daniel Boone discovered "Kentucky" and about the same time George Rogers Clark made his way westward from Virginia and down the Ohio River. At this time Kentucky, was open, uninhabited hunting ground by mutual agreement among the Indians. The population was at least two white men and a few Indians; filter plants in those days apparently were unnecessary.

The Lexington Water Company was started in 1884. The first filters were constructed in 1894 and have been progressively increased from the original 2 to the present 6.3 million gallons per day capacity. This latter figure includes only two of the proposed ten one million gallons per day filter units described in this article.

The design of these filter units has been based on the following figures.

Reservoir capacity no. 1 to no. 4 inclusive, m.g.	1,765
Area of reservoirs, acres	465
Area of water shed, square miles	12.7
Water shed owned by water company, acres	2,098
Present average demand, m.g.d.	5.0
Present filter capacity, m.g.d.	6.3
Eventual filter capacity, m.g.d.	12.3
Clear well capacity no. 1 and no. 2, m.g.	1
Present sedimentation basin capacity, m.g.	3.0
Low lift pump capacity, m.g.d.	20.5
High lift pump capacity, m.g.d.	21.5

It is of interest to note that with the present reservoir capacity of 1,765 m.g. and a present normal yearly demand of 1,825 m.g. that 95 percent of a full year's supply for the city is available in storage.

[1] Presented before the Kentucky-Tennessee Section meeting, January 24, 1930.

[2] Chief Operating Engineer, Lexington Water Company, Lexington, Ky.

[3] Chief Engineer, Community Water Service Company, New York, N. Y.

The original design, for the clear well and the filter units, as prepared by Caird and Wheeler of Troy, N. Y., provided for a clear well approximately 103 feet long, 56 feet 8 inches wide by 15 feet deep with a capacity of roundly 600,000 gallons. This new clear well in conjunction with the existing clear well with a capacity of 400,000 gallons was deemed ample to provide for variations in pumpage over filtration.

The clear well basin is divided into four main divisions as follows: Center division, into which all of the filters will discharge, has side walls, approximately 6 feet high, above the clear well floor above which openings communicate with the adjoining basins. The wash water supply is drawn from this central basin and provides an ample supply of water irrespective of the water level in the clear well proper. The basin across the front of the clear well connects directly at floor level with both of the adjoining side basins. A provision for cleaning is provided at the rear end of the center basin under the filters through the provision of a barrier wall approximately 6 inches high and a sump provided in this center basin. This clear well was constructed by contract in 1925 under supervision of Hugh R. Cramer, Chief Engineer.

Late in the fall of 1927 the Lexington Water Company was taken over by the Community Water Service Company. During the winter of 1928–29 plans for the filters which were to be superimposed on the new clear well were prepared. Construction was carried on by local water company forces under the direct supervision of Hugh R. Cramer. Work was started in April, 1929. Owing to changed local operating conditions the filter plans as originally prepared by Caird and Wheeler were revised and redrawn, although of course they were of necessity kept to the limits prescribed by the existing clear well. The first filter was placed in operation in August, 1929, five months after construction was started.

In redesigning these filter units several additional controlling factors were added to those already mentioned. The more interesting of these was the provision of the air wash as originally planned, the combining of the filter effluent, wash water, air wash and rewash through a special fitting, and the installation of cement lined cast iron laterals.

These laterals were 2-inch cement lined pipe with five $\frac{1}{32}$-inch openings lined with brass eyelets. These eyelets were inserted in the holes in the laterals before the cement lining was placed with the result that no exposed iron is in direct contact with the water.

Air in amount of 4 cubic feet per minute per square foot under pressure of 4 pounds per square inch was made available.

A pressure of 15 pounds to the square inch was used as a maximum for the wash water and sufficient volume and head was provided in the size and location of the wash water tank to produce this pressure. The wash water tank has sufficient capacity to provide for two complete filter washes.

The maximum rise of 16 inches per minute was used in the design of the wash water piping, although in the operation of these filters

FIG. 1. FIRST UNIT OF FILTER BUILDING COMPLETED EXCEPT FOR EARTH FILL

The superstructure houses two 1,000,000 gallon filter units and the head house. Two additional units, nos. 3 and 4, constructed outside of the superstructure, show at the left of the picture.

it is not anticipated that a rate of rise in excess of 10 or 12 inches will be required owing to the provision for air wash.

The most unique feature of this building is the construction of each filter as a separate and distinct unit. No bonds exist between adjacent units or between the units themselves and the supporting clear well. No waterproofing of any kind was used in concrete. This provision for flexible construction makes it possible to provide an excess of filter capacity over anticipated demand to meet the requirements of the city until about 1950 without an unduly large expenditure of money at this time.

From a construction standpoint the biggest advantage derived from this type of construction is the avoidance of temperature cracks and the location of expansion joints in the filter walls.

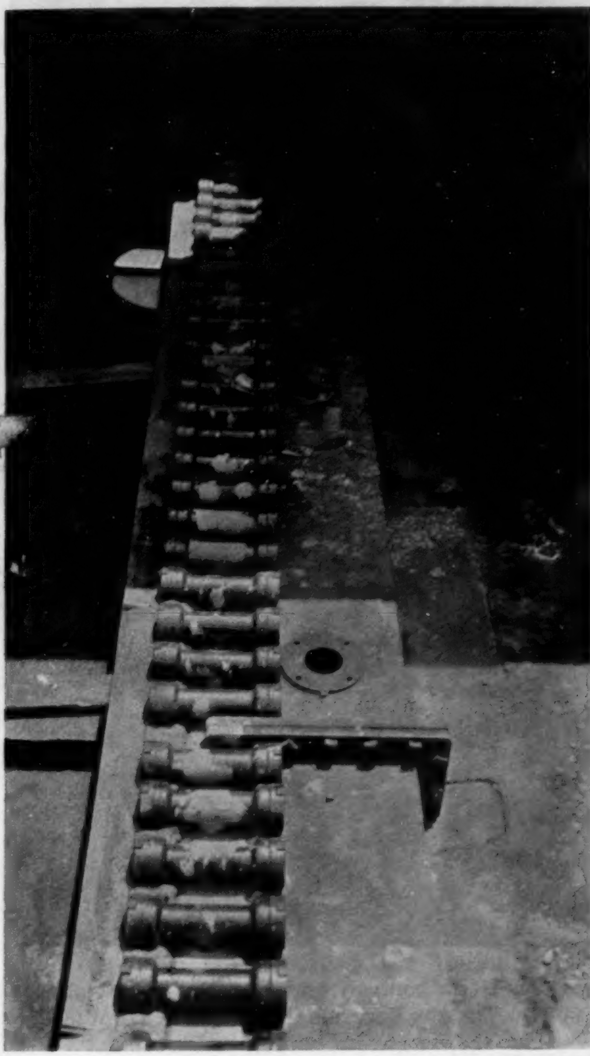

FIG. 2. CAST IRON MANIFOLD AND LATERAL TEES CONCRETED IN PLACE IN THE FILTER BOTTOM

FIG. 3. CONCRETE DUCT CARRYING 8-INCH AIR LINE, 4-INCH STEAM LINE AND HIGH AND LOW TENSION ELECTRIC CONDUITS FOR AIR WASH, HEAT AND POWER IN THE NEW FILTER BUILDING

This duct runs between the old pump station and the new filter building which is shown in the background.

While these filter units No. 1 and No. 2 were in successful operation, in August, 1929, work was far from complete. No superstructure had been built, operating tables were still in the course of construction and the filters at this time were what might be called "open air" type. The heavy demand caused by the drought in 1929, however, made it necessary to place these units in immediate service. These two units were started under supervision of Martin E. Flentje, Superintendent of Purification, Community Water Service Company, Harrisburg, Pa.

FIG. 4. VIEW SHOWING THE EQUAL DISTRIBUTION OF AIR THROUGHOUT THE FILTER

Air wash is used through the same lateral pipes and orifices as used for the water wash.

H. Cable Cramer of the Lexington Company was directly in charge of building of forms and placing of concrete, as well as all other building construction.

In constructing the new clear well a considerable amount of rock was excavated. This was removed from the immediate site of the work and subsequently crushed and prepared with regular plant equipment. All of the stone for concrete used in the filters was crushed, screened and washed on the job. All concrete for the filter boxes and head house was poured through the use of one concrete tower and chutes which greatly facilitate the placing of this material as the top of the forms were 12 to 14 feet above the ground surface.

The construction of the manifold and the method of placing the special cast iron lateral tees was such that a large air pocket is formed in the top of the manifold with all of the horizontal openings into the lateral tees set exactly level, insuring uniform distribution of air through all of the laterals. "Bosses" were cast on the top of the tees at either end, and the horizontal opening of the inlet on the bottom of the tee was ground to an exact distance from these bosses and parallel to their plane.

Cast iron laterals were rigidly supported at the far end and all laterals were checked for level before the joints were poured at the tees. Before gravel was placed in the filters water was admitted to the filter box to a depth of about 6 inches over the lateral pipes. A slight air pressure was carefully applied to the lateral system and extreme care was taken to note that a uniform amount of air showed at each and every opening in the bottom of the lateral pipe and that the air bubbled up on each side of the lateral pipe indicating that the hole was exactly on the bottom. Coarse gravel was carefully worked in between and under all the lateral pipes for support and carefully levelled over the top of same.

Gravel placed in these filters was Louisville gravel purchased unscreened. Gravel was carefully graded and washed on the job before being placed in the filters. The following sizes and depths of gravel were used in these filter units.

Depth, inches	*Size, inch*
6	$1\frac{1}{2}$ to $\frac{3}{4}$
3	$\frac{3}{4}$ to $\frac{1}{2}$
2	$\frac{1}{2}$ to $\frac{5}{16}$
2	$\frac{5}{16}$ to $\frac{1}{8}$ and smaller

Owing to limited electrical capacity it was decided to operate the air blower by steam and accordingly it was placed adjacent to the boiler room in the pump station. An 8-inch air main, a 3-inch steam main for heating and electric conduits for light and power in the filter house were carried from the pump station to the new filter building in the concerete conduit as shown. This conduit was later covered with a concrete slab with manholes over flanges.

All local construction work was carried on by water works employees and the best possible use was made of mechanical equipment.

The foundation for the wash water tank is known as the ring type foundation.

The following equipment was used in the construction of this filter.

1. *Wash Water Tank*—Capacity 49,000 gallons. Built by Pittsburgh Des Moines Steel Company, 24 feet in diameter, 14 feet 10 inches high.

Painted by company employees. Inside three coats Goheen Corporation's Highway Red No. 21. Outside one coat Highway Red No. 21. One coat Detroit-Graphite Company's undercoated No. 323. Finish one coat Detroit Graphite Company's Silver-Cool No. 325.

2. *Wash Water Pipe Line from Tank to Filter*—16-inch cast iron pipe, cement lined.
3. *Wash Water Pump*—Allis-Chalmers. Capacity 500 g.p.m., 51-foot head, 74 percent efficiency. Motor drive 10 h.p., 3500 r.p.m., 3-phase, 60-cycle, 220 volts, Automatic General Electric Compensator controlled by
4. *Wash Water Tank Control*—Bristol Company's Model No. 80, Scale 0 to 100 feet with high and low adjustable contacts.
5. *Clear Well Level Indicator and Alarm*—Bristol Company's Model No. 80, Scale 0 to 25 feet mounted in the head house to indicate the clear well water level. This instrument has high and low alarm.
6. *Wash Water By-Pass*—This connection provides for the filling of the wash water tank direct from the high pressure system through an altitude valve.
7. *Wash Water Venturi Meter*—Builders Iron Foundry Type "NS" size 14 by 7 with Type "Y" Register—Recorder graduated to indicate flow and also the rate of rise of wash water in inches per minute.
8. *Wash Water Troughs*—Built by Vogt Brothers, Louisville, Ky.
9. *Liquid Level Controller on Influent*—Builders Iron Foundry Company.
10. *Steel Influent Riser*—Pipe 3 feet 6 inches in diameter with 36-inch horizontal section with connections to filters. Sotter Brothers, Pottstown, Pa.
11. *Rate Controllers*—Builders Iron Foundry 10-inch, direct acting 500,000 to 2 m.g.d. rate.
12. *Loss of Head Indicator*—Builders Iron Foundry Type "I" operated by diaphragm—pendulum type control.
13. *Air Blower*—R. H. and B. F. Roots, Connersville, Indiana No. 3 Horizontal, top discharge, $\frac{5}{35}$ cubic feet per revolution, 1500 cubic feet per minute at 4 pounds maximum discharge. Driven by 9- by 9-inch Troy, vertical enclosed self-oiling engine with special cylinder for the high and low steam pressures available in the pump station Engine equipped with diaphragm governor control.
14. *Operating Table Tops*—Asbestos Ebony, Jones-Manville Company.
15. *Table Frames and Laboratory Tables*—Combs Lumbers Company.
16. *Operating Handles and Indicators*—International Filter Company.
17. *Four Way Valves*—Lunkenheimer, $\frac{3}{4}$-inch packed.
18. *Hydraulic Valves*—Rensselaer Valve Company.
19. *Manifolds and Laterals and Special Filter Casting*—McWane Cast Iron Pipe Company.
20. *Steel Frame and Reenforcing*—Supplied by Jos. T. Ryerson and Sons. Building constructed of best grade common brick with Indiana Limestone.
21. *Steel Sash*—Furnished by Glenfire Steel Company.
22. *Gypsum Pyrobar Roof Slab*—U. S. Gypsum Company.
23. *Switchboard and Electrical Equipment*—General Electric Company.
24. *Lead Covered High and Low Tension Wiring*—Simplex Wire and Cable Company.

DAYTONA BEACH, FLORIDA, WATER WORKS[1]

By B. F. Tippins[2] and F. E. Stuart[3]

Daytona Beach is the first Florida city to own and operate a water softening plant for domestic purposes. The original water works were designed and built in 1908 by C. M. Rogers, consulting engineer of this city. The first unit of the softening plant was built in 1910 by the city, successive units being built along with the requirements.

Boom days hastened consolidation of three small towns into the city of Daytona Beach with a population of 25,000, and owning three separate water softening plants all of different types.

SOURCE OF SUPPLY

Artesian wells are the source of supply. There are seventeen 6-inch wells, varying in depth from 180 to 240 feet, and have an approximate head of 6 feet above sea level, which is drawn down to approximately 5 feet below, by heavy loads.

A contemplated new plant for the entire city will be built on the mainland with several lines leading across the river to elevated storage tanks. The length of the peninsula requires use of two such tanks to keep a sufficient constant pressure in outlying districts.

Water is brought to aerators by centrifugal pumps, varying in size from 4 to 10 inches, with capacities of 700 to 3,400 gallons per minute. Wells have a good supply at all seasons.

CHEMICALS USED

Lime-soda treatment is used in all of the softening plants. Other methods have been tried, but, due to high cost, have been discontinued. The water is so hard and has such high mineral content that 16 grains per gallon of lime are used at one of the plants.

Treated water, delivered to city mains in 1929, totaled 524,929,000 gallons, a daily pumping rate of 1,438,000 gallons. The per capita consumption was 67 gallons daily.

[1] Presented before the Florida Section meeting, April 11, 1930.

[2] Superintendent, Water Works, Daytona Beach, Fla.

[3] Chemist, Water Works, Daytona Beach, Fla.

The chemicals used for the year were:

	pounds	*p.p.m.*
Lime	998,902	208
Alum	53,127	11.9
Soda ash	31,475	6.8

The treated water retails for 20 cents a thousand gallons, and costs, above all overhead, 10.5 cents to pump, treat and deliver to a consumer.

The water works has experimented with all well-known makes of softening reagents and concluded that the lime-soda-alum treatment is about the cheapest and most satisfactory method of softening. There are several methods that will give equal or better results, but their cost is prohibitive.

METHOD OF SOFTENING

The softening system of the Mainland station is intermittent, while that on the Peninsula is continuous. Water on the Mainland is pumped through an aerator located about 4 feet above a 75,000 gallon reservoir. The water falls over a splash board into a series of baffle-plates causing a fine spray, which exposure releases sulphuretted hydrogen in the air.

When the reservoir becomes filled an air agitation system, through a net-work of perforated pipes, is turned on. This method of agitation is used because it was the original equipment although mechanical agitation is the best. The new plant will have mechanical agitators and recarbonating equipment. When air pressure has the entire body of water in motion, the milk of lime-soda solution is added and the whole is agitated for about twenty minutes, when powdered alum is scattered on the surface and allowed about three minutes to mix into solution. Too much agitation after application of the coagulant will break up the floc and delay the settling.

Less than five hours after treatment starts, including a settling period, treated water is ready to be delivered to the mains, by an original decantation process used in emptying reservoirs. There are nine tanks, similar to the one mentioned, in which treatment is identical. They are used in rotation and become "ground-storage" as long as they are not being used for treatment, providing a total of 675,000 gallons of water in reserve in case of unusual demand. "Clear-well" storage to pumps has a capacity of 60,000 gallons additional "ground-storage."

In the continuous lime treating process employed in the Peninsula plant, water flows to centrifugal pumps located in a "low-head" house which was specially constructed for conditions encountered with a water table 6 feet from the surface. This concrete well-pit was built to house raw water pumps and manifold located 8 feet below the surface. In this installation there is no need of priming pumps.

Seven 6-inch wells connect directly to a large manifold so designed and constructed that any portion of the well system may be shut off without altering operation of the plant. Well pumps operate under a head of 70 feet flowing to aerators where water falls into a "downtake." Here the lime-soda solution, pumped from a location in chemical house, is applied. Alum solution enters this same downtake in dilute solution and under pressure.

The water enters pressure filters which have a combined capacity of 1,200 gallons per minute, and then flows to the ground water storage reservoir. The total ground storage capacity is 410,000 gallons. From this reservoir water is pumped to the city distribution system by three high pressure pumps, varying in size from 400 to 800 gallons per minute, all electrically operated by Cutler-Hammer control.

The elevated storage on the Peninsula consists of 135,000 gallons in two tanks with 75,000 on the Mainland. The total elevated storage capacity for the city is 210,000 gallons, which will maintain a normal pressure of 55 pounds.

DISTRIBUTION SYSTEM

In the distribution system there are 64 miles of cast iron water mains, varying in size from 4 to 16 inches. There are 464 hydrants and 612 valves, giving an average of 7 hydrants and 9 valves to the mile.

There are 4,850 services varying in size from $\frac{3}{4}$ to 6 inches. Sprinkler connections for fire prevention number 23. All services are metered, giving an accurate estimate of total revenue. Approximately 80 percent of all water pumped is accounted for in dollars and cents. Hydrants used for construction purposes are equipped with special meter attachments.

All tests made on water meters show they are very accurate, varying less than 2 percent on the average. As a general rule they are about 1.5 percent slow. Meters of the disc type with enclosed train are used for small services and compound meters are used for large

services. No cross connections are used in the distribution system. Hot water service connections are required to have a check valve and a relief valve.

Services are brought to curbs in cast iron meter boxes and equipped with curb cocks and service valves.

The odor of raw water is caused by compounds of sulfur in solution which can be relieved entirely by aeration and other treatment.

TABLE 1

Chemical properties of Daytona Beach water

	RAW WATER	TREATED WATER
pH	7.4	8.4
Total hardness	340	119
Carbonate hardness	300	70
Non-carbonate hardness	40	49
Bi-carbonates Ca and Mg	300	26
Normal carbonates Ca and Mg		44
Normal carbonates Na_2CO_3		00
Caustic alkalinity as $Ca(HO)_2$		00
Caustic alkalinity as NaOH		00
Total chlorides	170	170

BACTERIAL PROPERTIES

The raw water is clear and bacterially safe. The only case of contamination in our treated water was in the summer of 1928 when a high count and slight fermentation caused uneasiness. This was traced to the use of a section of fire hose which the street department had employed to flush a storm sewer without our knowledge. Removal of this hose and a thorough cleaning of equipment remedied the matter and no such experience has ever recurred.

The average count for raw water for 1929 was 60 bacteria per cubic centimeter. There were no colon organisms.

The average count for treated water for 1929 was 20 bacteria per cubic centimeter. Colon organisms have never been in the lime treated water. Very few cases of typhoid are reported in this section, and they are readily traced to contaminated oysters. Due to the strict enforcement of sanitary laws by the State Board of Health in regulating oyster handling, no typhoid from this source occurred last year.

The water works has its own chemical and bacteriological laboratory which is essential to proper control of a modern water plant. The entire softening system functions through this laboratory at all times in a positively controlled and satisfactory manner.

Experience at Daytona Beach has been that laboratory supervision and rigid control have effected pronounced savings in operation of its water works, besides assuring efficiency of service under the highest sanitary conditions.

A NEW SOURCE SUPPLY FOR ORLANDO, FLORIDA[1]

By C. F. Underhill[2]

During the past ten years, the State of Florida has experienced a wonderful growth and development. As we all know, the direct cause of the growth of any community is an increase in population. With an increase of population, there naturally follows an increase in business, in the building of new roads for business and in the settlement of more territory for the development of more business. For the water works men it means an increase in the use of water. This phenomenal growth in the state was, of course, expected by the real estate men, as may be seen by taking a ride over almost any country road where you may see subdivision entrance decorations, which should have a life of at least twenty years, already falling into decay. The rank and file of the people, including the water works men, expected a growth, but this was to be a natural, gradual, and healthy one which would not reach too great proportions too quickly or before the customary expansion programs had been completed.

The citizens of Orlando had anticipated a natural growth and had started, in 1922, to build a light and water plant to take care of any future increase in population. The consulting engineers had plotted the usual curves showing past, present and expected future population and had designed a plant which would be sufficient for the city's needs until at least 1940, before which time new plans could be made for any further expansion. Ample time was thus given to make provision for the cost of this extension which could have been made by setting aside a reserve fund from the profits of the operation of the original plant over a fairly long period of time. These later plans could be slowly and carefully worked out so as to make the plant capable of supplying the needs of the city for many years to come.

Contrary to expectations, however, the growth was not a natural one, but seemed to be the confirmation of a realtor's dream of building a city overnight. In 1926, four years after the first plans were

[1] Presented before the Florida Section meeting, April 11, 1930.

[2] Orlando Utilities Commission, Orlando, Fla.

made, it was discovered that the population was increasing so fast that before 1930 it would reach the figures which had previously been estimated for 1940. This called for some serious and rapid thinking and planning by the Utilities Commission to obtain a further water supply. The network of electric power lines, which was stretched all through the state, would have allowed a temporary emergency connection to have been made for electricity, but there was no such underground network of water pipes stretching from town to town. We could not tap a main put in a meter and get some more water while making plans for a new supply. The new supply must be obtained quickly, since the need was urgent and also it must be obtained as cheaply as possible, since the expansion reserve fund had not had much time in which to accumulate.

SEARCH FOR NEW SUPPLY

Orlando and its vicinity has always been blessed with an abundance of lakes, and from the time that the first privately owned water works was put in operation, the supply has been obtained from lakes. Therefore, it was only natural that any new supply should come from some of the other lakes which were not in use. There are twenty-seven lakes within the city limits and fully as many more within a 3-mile radius of the city limits, so it was only a question of which lake or chain of lakes could be used most advantageously. The supply ought to have all the desirable qualities, if possible, but with the added drawback that it must be obtained quickly and cheaply.

As a strange coincidence, just before it was decided to look for the raw supply, the city had purchased a large tract of land to be used as the municipal airport. The land borders on a fairly large lake about 1 mile west of the city limits and about 3.5 miles from the water plant. I would like to add, incidentally, that by another strange coincidence, this lake has always been known as Lake Underhill, which is the name by which the author of this paper has always been known. However, the author does not wish to infer that the excellent name influenced the choice of Lake Underhill as a water supply. Everybody knows that enough other good names also came over in the good ship Mayflower to sink it three or four times. The airport naturally called attention to the lake and a quick survey showed that the airport shoreline included about one-fourth of the total shoreline and a new county road extended along another fourth. Here was one-quarter of the lake shoreline absolutely under city control and

one-quarter partially controlled by the county. This made the lake interesting enough for further study, so a complete survey was made with a view of the ultimate use of the lake as the new supply.

LAKE UNDERHILL SUPPLY

Lake Underhill is approximately circular in form, with a diameter of about 3500 feet, the shoreline area at high water being around 175 acres. It has an average depth of 20 feet, the greatest depth being 33 feet. This gives a storage of 1,100,000,000 gallons and the slope of the lake bottom is such that the top 7 feet will pump about 400,000,000 gallons, or one-third of the total storage. The lake even at the 7-foot stage, has an area of about 165 acres, and will still have a very attractive appearance in spite of being drawn down to this depth. A quarter of a mile south of the lake are two more small lakes which are evidently connected to the large lake by underground passages since the lake levels in all three lakes always remain at the same height. These two lakes hold about 150,000,000 gallons each and tend to make up whatever evaporation there may be from Lake Underhill. The drainage basin of this section comprises an area of about 3.5 square miles, the lake making 8 percent of this. With an average annual rainfall of 51 inches, it is probable that Lake Underhill alone will be able to add enough water to the main supply so that no further expansion will be necessary for some years. Assuming a population of 50,000 and a per capita consumption of 85 gallons of water per day, it would take about 1.5 billion gallons to supply the wants of a city of this size. This amount is about 0.5 billion gallons less than the old and new supply combined.

A study of the quality of the water in the lake showed that this water is excellent. The bottom of the lake is made of a hard white sand with practically no vegetable growth. This apparently makes the water clear, colorless and sparkling in its raw state and, therefore, it needs no complicated or troublesome color-removal treatment. The total alkalinity is 8 p.p.m., the pH value about 7.4 and the soap hardness about 20 p.p.m. This natural alkalinity responds readily to a treatment of one-half grain of alum per gallon, this small amount giving a heavy floc. The floc end-point is not very delicate and large variations in the alum dosage are not troublesome, except from an economical standpoint. The softness allows us to bring the pH of the filtered water up to about 8.5 by the addition of 0.5 g.p.g. of lime without raising the hardness to any great extent. The algae in

the lake water are readily held under control by an occasional dosing with two or three barrels of bluestone. Thus we see that this new supply is very acceptable as to quality and ease of treatment.

The question of present and possible future pollution of the supply was partially answered for us by the fact that, as stated before, the airport and the county road accounted for about one-half of the shoreline. This was certainly a great help towards acquiring control of the remainder of the lake shore and also the land in the lake itself. With the purchase of the airport the city also obtained over one-quarter of the lake proper. The rest of the lake bottom was owned by only four different people and all but one of these readily gave the city deeds for their holdings. The holdout owner who really only had a tax title and who also was the only one who did not own a lake shore front, proved very stubborn, especially when, by mistake, we filled in a triangular piece about 100 feet on a side and thus made him a lake front lot right in the center shoreline of the airport. This was too great a temptation for him and the Utilities Commission was forced to carry the case to court, where it was settled for a nominal sum. This was the only litigation that was necessary to acquire all the lake bottom and the lake shore.

The lake shore not already controlled by the city or county was owned by one large subdivision company which had sold several lake front lots, and by seven other large holders who had not subdivided. The Commission already had a strong argument for convincing these lake front owners that it was to their benefit to give up their riparian rights to the city. There happened to be, in the lake, a large dredge which had been used by the original owner of the airport land to smooth out his lake front. He turned this over to the city with his land and the city in turn loaned it to the Commission. The Commission agreed in writing to fill in all the lake front so that there should be a strip of land not less than 60 feet wide between the water and abutting property. This was to be laid out at some future time as a broad boulevard and parkway and no structure was to be permitted between the lake and the property. The boulevard and parkway were to be beautified with grass, palms and shrubbery and would, therefore, make the property as valuable as though it had riparian rights. In some cases, the property now fronts on the county road and on the future lake boulevard and some of the subdivision lots are much longer than originally planned, due to the fact that it was necessary to fill in the lake more at some points than origi-

nally planned. The dredging was started at once, has been completed and the dredge has been taken out of the lake and over one thousand palms have been set out on the boundary of the contemplated boulevard. This made it somewhat easier to get control of the lake front, but you may be sure that it took a whole lot of diplomacy, private conferences, trading, and, of course, some monetary payments to complete the job, especially since the negotiations were made during the so-called boom period. All complications were satisfactorily smoothed out by the Commission's able General Manager, Mr. A. P. Michaels, whom you all know, and now the whole lake front and land in the lake is completely in the hands of the Commission, thus safeguarding the source of supply from pollution.

While a water supply must be had at any cost, still some study must be made to keep this cost as low as possible and at the same time furnish an adequate supply. In this case also, the time element entered in, due to the unexpected heavy demand of the large increase of population. A careful survey showed that the Lake Underhill proposition was the best at the time. The lake elevation is about 25 feet higher than the regular supply lake, making a gravity feed apparently possible. However, a further study showed that the transmission main would have to be laid in some stretches, to a depth of 12 and 15 feet, thus increasing the cost and at the same time, the flow from Lake Underhill would have to be pumped from the main lake anyhow. Therefore, it was decided to put in a pump at Lake Underhill and pump directly to the aerators at the plant, the increased cost of pumping at a higher head being compensated over a period of years by the lower original cost of installing the main.

The installation at present consists of a stucco-on-hollow-tile pump house with concrete foundations about 10 feet below ground. This houses a 6 m.g.d. DeLaval centrifugal pump direct connected to a 150 H.P. synchronous Westinghouse motor with remote control at the main plant, 3.5 miles away. The pump suction is about 7 feet below high water so that no priming will be necessary for some time. We have two impellers, one of 3 m.g.d. capacity and the other of 6 m.g.d. which we interchange as the load changes from summer to winter. The transmission main consists of about 17,000 feet of 20-inch bell and spigot cast iron pipe, Class "B." Class "A" pipe could have been used just as well and at less expense, but it was thought that, at some future time, this pipe might be needed for higher pressures and so Class "B" pipe was installed.

The following figures will give an idea of the cost of acquiring and developing the new supply and may possibly be of interest:

	dollars
Preliminary engineering	254.09
Cost of land and water rights	40,950.00
Dredging and filling in	19,989.39
Pump house	5,742.09
Pump, motor, etc.	3,743.44
Suction line and intake	3,581.56
Transmission main	76,987.18
Beautification	4,247.01
Total cost (approximate)	155,494.76

FUTURE SOURCE

In conclusion, I would like to mention the fact, that about 4 miles away from Lake Underhill, there is another lake with an area of over 1800 acres. This lake is a potential water supply for a fairly large city. While it is not to be expected that complete control of the lake can be obtained, still some regulations can be made in order to protect the supply from excessive pollution. This water can readily be pumped to Lake Underhill as a storage reservoir, and this storage will tend to cut down the pollution to about where it will not be too hard to eliminate by treatment, say prechlorination, etc. Thus Lake Underhill may be used in future years, after serving out its present usefulness, as a link in Orlando's future supply system.

THE WATER SUPPLY OF SCHENECTADY[1]

By Warren C. Taylor[2]

The first attempts at a city water supply in Schenectady were to utilize springs at the edge of woods above the town and to conduct the water through wooden log pipes to the city. The connections were made by iron ferrules. Union College at one time, in 1840, had a private supply conducted through a 1-inch glass pipe surrounded with 2 inches of concrete.

The first real water system was completed in 1871 when a private company was organized. The water was pumped from the Mohawk River. A well 6 feet wide and 114 feet long was constructed near the edge of the river. The walls of the well were brick and the top was a brick arch. The water from the river filtered through the sand and gravel on the outside of the walls into the well.

As the demand from the city became too great and difficulties in maintenance of pipes arose, the city took over the company in 1885 and proceeded to enlarge and improve the system. Two intakes into the river were laid. The one located at one of the piers near the Scotia end of bridge over the Mohawk, had a 24-inch supply pipe and the other in the river below the pump house, at the end of Washington Avenue had a 20-inch intake pipe. This, with the enlarged distribution system, improved conditions greatly.

While this supply remained adequate for several years, yet the towns up the river developed rapidly and proceeded to empty their sewage and wastes into the river so that great danger from pollution arose. This came to a crisis in 1890–1891, when a serious epidemic of typhoid fever broke out in Schenectady. This lasted from July, 1890, to April, 1891. About 300 cases were reported. Doubtless there were many more, because compulsory reporting was not enforced in those days. Schenectady had a population at that time of about 20,000. The sewage of Schenectady was carried into the same river. This resulted in an epidemic in some of the cities down the

[1] Presented before the New York Section meeting, October 3, 1929.

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Mohawk which used the river for their supply, notably Cohoes which reported over 1000 cases. West Troy also took its supply from Mohawk and reported over 100 cases.

The Hudson River received the waters of the Mohawk at Cohoes and then in a few miles flowed past Albany. Here the epidemic was quite serious, resulting in several hundred cases of fever and nearly 100 deaths.

For Schenectady a new supply was imperative and a thorough search was made for a new source. Several lakes in the vicinity were suggested. These were from 10 to 25 miles distant. Prominent citizens of the neighboring town of Rotterdam were finally successful in convincing the city authorities that in their land about 2 miles from the city ample water could be obtained by dug wells. Their contention was that there was an underground stream under this area. Investigation confirmed this allegation. A supply amply abundant and of remarkable purity was discovered.

A well 60 feet long by 8 feet wide and 43 feet was constructed. The walls were cut stone, the lower courses laid with open joints. The roof was an arch about 20 feet below the surface of the ground. In the center was a manhole opening which gave access for inspection. The suction pipes from the well were two 24-inch pipes. These two suction pipes were also connected with two 24-inch pipes which led to an intake in the Mohawk River. This was evidently an emergency provision to relieve the fears of those who suspected the adequacy of the underground supply. An occasion to use these intakes never arose, but they later became the cause of an unfortunate experience.

A 24-inch main was laid to the city. A 1,000,000-gallon tank was located at Albany Street, for emergency purposes. The city boasted, at this time, as the possessor of one of the finest systems in the country.

The constant growth of the city created a greater demand and in 1904 two additional wells were added, a new pumping station and outfit were also constructed. A new 36-inch riveted steel main conducted the water to the city, and to another tower located at Rugby Road. This tower had a capacity of 2,000,000 gallons.

Ten years later, 1914, the two water towers were torn down and replaced by a concrete reservoir on Bevis Hill at the end of the system. This has a capacity of 20,000,000 gallons. An additional 36-inch pipe was laid to the city. The pumping station was entirely rebuilt.

Of interest to engineers came the necessity in 1922 of replacing

the 36-inch steel conduit, laid in 1904, with a cast iron pipe. This steel pipe was found badly corroded. It took two or three years to accomplish this replacement. In order to meet the requirement of the electric power company supplying electricity for the pumps, the entire installation of pumps was also revised.

Without going into detail further as to this development let me describe briefly the plant as it is today.

PRESENT SUPPLY

The source of supply includes three open collecting wells located at Rotterdam about 2 miles from Schenectady. These wells are of following sizes:

No. 1, 8 feet by 60 feet by 43 feet deep
No. 2, 47 feet diameter by 44 feet deep
No. 3, 47 feet diameter by 40 feet deep

The wells are connected to each other by 20-inch siphons.

The water is conducted to the city by direct pumping. At the end of the system is a 20,000,000-gallon equalizing reservoir. This reservoir is located on Bevis Hill. The construction is concrete, 520 feet by 260 feet by 22 feet deep inside dimensions. On the outside of the walls is an earth embankment and over the roof is 2 feet of earth.

The pumping equipment consists of 3 motor driven turbine centrifugal pumps, with a total capacity of 42,000,000 gallons per day.

All the water is measured by Venturi Meters.

The total consumption, on a basis of 104,000 population, is 142 gallons per capita per day. The average daily consumption is 14,870,000 gallons.

The land including the wells and pumping station owned by the city was only 5 acres. With the increasing encroachment of residences upon the surrounding territory and the attendant likelihood to contamination upon the water shed there arose a feeling among city authorities of the need of buying more territory surrounding the plant. Accordingly in 1925, 38 acres adjacent were acquired. Not only was this to safeguard the supply, but also to allow for future expansion. In the next year, 1926, through the influence of the Chamber of Commerce this area was further increased to 77 acres.

For some time the question of just how large this ground water supply was and whether it could be depended upon for future growth

had caused some anxiety to the city. The knowledge that a neighboring city was considering locating their source of supply close by and drawing from the same underground stream aroused Schenectady to action. Although the proposition was turned aside, the need for an answer to the question as to the size of the supply became more strongly emphasized.

SURVEY OF UNDERGROUND SOURCES

The outcome of the agitation was to make a small appropriation to conduct a survey of the conditions surrounding the well and to determine the real amount of the underground water available. To do this Dr. J. H. Stoller was engaged as geologist and the city engineering department supervised the engineering for the project.

All available means were used to determine the depth and area of the gravel deposits which were pierced by the public wells, with the following observations.

1. At some places the Mohawk River has cut through the strata and revealed the gravel. In one place a thickness of 50 feet is exposed.

2. Large gravel pits have been opened up at a few places and the gravel has been excavated for a depth of over 50 feet.

3. Borings by the State in connection with Barge Canal and Great Western Gateway were helpful. Some private wells had been drilled through the gravel. Some of these drillings have been 200 to 300 feet deep to rock and have shown the thickness of gravel layer through which they bored.

The borings by the State did not go to rock, although a depth of 170 feet was reached. In addition to all the available data that could be found, Dr. Stoller added valuable geological information from his study of this locality extending over many years.

4. Two borings for the dam at lock No. 8 of Barge Canal also proved of great value in determining the cross section across the river at the well location.

The city undertook with this beginning to make a series of borings in the vicinity of the wells. These were driven to bed rock and a careful log of each hole was taken. From these records a great deal of valuable knowledge was gained. Sections and profiles were plotted.

Some of the following conclusions reached may prove of value:

1. A most important conclusion is the extent in area of the water bearing formation. Let me quote a paragraph from Dr. Stoller's report:

The broad basin of the Mohawk west and northwest of Schenectady forms the surface of a mass of sand and gravel deposits. In diminished width the body of deposits extends up the river as far as Hoffmans and down river to near Rexford. Along the valley flats and on the islands of the river west of Schenectady the gravels are covered by an alluvial deposit, or loamy soil. North of the river and west of Scotia the surface rises about 70 feet above the valley flats and the materials are predominately coarse gravel. Several large gravel pits are developed in this quarter. Traced to the northwest the materials of the valley filling become less coarse but the sand and gravel formation is continuous to Hoffmans, where a rock barrier across the valley marks its western limit. A similar rock barrier at Rexford marks the eastern limit.

The areal extent of this sand and gravel formation is approximately 15.25 square miles. The wider portion of the basin extending westward from Schenectady to where the valley narrows west of Scotia is approximately 7 square miles.

2. The thickness of the sand and gravel layers in hole No. 1 near the wells was 110 feet. In others it varied. For instance, in two of the wells it was 12 and 22 feet. After careful study of all the borings an average thickness of 66 feet was considered as a reasonable estimate.

In all the cross sections, the valley showed impervious hard pan of blue clay resting on bed rock, upon which the gravel rested. In some of the holes the hard pan was missing. These holes were located at the upper end of the formation. Of interest to the geologist was the elevation of the rock bottom which checked with elevation of the bottom of the river where the Mohawk joined the Hudson, quite confirming the contention that all this basin was at one time a great lake, Albany basin, which through subsidence, withdrawal of ice sheet and wind blown sands, had resulted in the present formation. The old basin had filled up on the rock with clay from erosion of the local rocks, while the great ice sheets had later deposited their rocks and gravels from the igneous rocks above this location. The rock barriers at Rexford on south and Hoffmans on north had held these deposits within these bounds.

With the area of 15.2 square miles for the extent of this gravel and 66 feet as average depth the volume can be determined.

As a water bearing formation its porosity would also be needed. It seemed impossible to determine this accurately due to great variation in structure throughout the mass. That it was in most cases found to be very porous was demonstrated by the freedom with which water passed through it. This gave the geologist the assurance that 30 percent was a conservative percentage to use. Undoubtedly this is conservative. Using this 30 percent and 15.2 square miles area

and 66 feet thick a large underground reservoir of capacity of 63,000,000,000 gallons is available. The city is pumping about 5,500,000,000 gallons per year from this reservoir.

3. It is one thing to have a reservoir sufficiently large, it is quite another to keep it full of water.

AVAILABLE UNDERGROUND WATER

The rainfall for Schenectady according to the rainfall chart made out some years ago by Conservation Commission of New York State is about 42 inches a year. The average for the State of New York is 38 inches per year. The rainfall records for the past few years taken at Schenectady Sewage Disposal Plant show about 38 inches. Of this rainfall, the amount available is the precipitation minus the losses due to evaporation and transpiration. For this locality the average temperature is about 47°. The corresponding transpiration loss is from 7 to 9 inches. For the conditions of soil and topography a value of 7.5 inches might be safely assumed. The evaporation in most of the results given by Meyer is double the transpiration or, in this case, 15 inches, leaving available 38 – 22.5 or 15.5 inches rain per year. There are other losses due to interception of private wells and sewers and other uses which might be estimated as 1.5 inches, giving 14 inches as the total.

This would seem a fair estimate for rain falling on the surface directly above the gravel bed area, but of the rain coming into this gravel from the surrounding water shed doubtless more would be intercepted. As the hillsides also are rockier and steeper the evaporation factor would probably be less. If then the transpiration is considered as 7 inches and evaporation 13 inches there is left 38, 20 or 18 inches available. This also corresponds to a run off expected for a stream in this locality of about 50 percent. To estimate the amount of this going into gravel is a pure guess, owing to the amount of interception. If this is estimated at 50 percent then the total amount available is about 9 inches.

The area overlaying the gravel bed is 15.2 square miles. This for 14 inches of water gives a total of 3,700,000,000 gallons per year, and area of water shed exclusive of gravel area is 35.5 at 9 inches per year gives 5,500,000,000 gallons, thus affording a total replenishment of 9,200,000,000 gallons per year as against 5,500,000,000 gallons used by the city during 1927.

CIRCLES OF INFLUENCE OF WELLS

Other interesting data derived from these borings showed that the blue clay which underlay the gravel formation sloped from the hills to the river. This resulted from a study of several profiles and sections plotted from the logs of the borings.

A partially completed study was made from which the circle of influence could be determined. This consisted in a series of readings of the water levels in the borings and the wells after various pumping conditions.

One study was made June 1, 1928. The pumps were shut down at 7 a.m. At 10 a.m., three hours after, it was found that the waters in the wells and borings were practically the same, indicating that in three hours after continuous pumping the cone of depression had filled up and water in the strata had become normal.

On June 28 pumps were stopped after fifteen hours pumping and readings taken eight hours after this. Equilibrium had been established by this time. On the next day readings were taken after fifteen hours pumping in all the holes in order to determine the extent of the effect of draft of water by pumps. Plotting these and drawing contour lines indicated a most irregular area affected varying from 830 feet away from the wells to 150 feet.

The data are too incomplete to go further with this study. Some time in the future more complete data should be obtained which will bring valuable results in regard to circle of influence due to pumping.

Further study ought also to be made in regard to underground flow, and some method should be used actually to measure velocity. It was hoped at this time that the Slichter method could be used here, but the appropriation was not sufficiently large to include further study along this line.

POSSIBLE EFFECT OF MOHAWK RIVER

The close proximity of the Mohawk River has always been a source of discussion in connection with the water supply. This has been the cause of anxiety from a sanitary standpoint in at least one experience.

In the spring of 1920 an epidemic of gastro-enteritis broke out in Schenectady. The cause was traced to the water supply. The State Department of Health made an analysis of the water from the city wells and found a noticeable turbidity and also B. coli present in 0.10

cc. samples. This brought about a thorough investigation of the supply.

In the original construction of the plant there had been two 24-inch mains which led through the well No. 1 into the River. This, as has been mentioned previously, had been done as an emergency for fear the well supply would not be sufficient. Quoting from the report at that time by Mr. Horton, Sanitary Engineer of Department of Health, State of New York:

Originally, the pump house now in use contained two large steam-driven reciprocating pumps which were connected to well No. 1 and were also provided with two 24-inch suctions extending to the Mohawk River. These two suctions passed out through the wall of the basement of the pump house about 18 feet below the surface of the ground, extended through two converging pipe galleries to well No. 1, crossed through the raised central section of that well, mentioned above, and continued in two parallel pipe galleries to a manhole at the road about 30 feet north of the well. From this point the pipes extended through the ground without galleries to the river. The galleries are about 6 feet across and 8 feet high. The walls and arched roofs are constructed of brick. The galleries are not paved, the bottom being formed by the gravel encountered in excavating them. Two manholes, one on the south side of well No. 1 and the other at the side of the road about 30 feet north of the well, afford access to the galleries. The suction pipes had been removed from the galleries, and the holes through the walls of the well and through the wall of the basement of the pump house had been sealed with concrete and brick. The portions of the pipes from the gallery to the river still remained in place. One of these pipes, the westerly one, was sealed with concrete. The other pipe, the easterly one, was apparently open from the river to the gallery.

The existence of these galleries and the fact that they could be reached by manholes, the covers of which were made visible by the melting of the snow, became known to the engineers from the Division of Sanitary Engineering of the State Department of Health on March 24, and they were immediately inspected. The galleries between the well and the road had apparently been full of water carrying considerable suspended matter a short time previous to the inspection. The bottoms were covered with a slimy deposit of black silt to a depth of from 0.5 to 2 inches, and the upper surfaces of all projections of the brick work to a height of about 8 feet above the bottom of the gallery were likewise covered with a deposit of the same material, the quantity of deposit being less on the higher projections. The deposit of sediment was practically uniform over the bottom of the galleries, except at points in both galleries about 10 feet from the north wall of the well. Here, in each gallery, there were several holes from 6 to 10 inches across on the top and extending from 1 foot to 2 feet down into the gravel in which the stones were perfectly clean, as if a swift stream had passed down through the coarse gravel at these points and carried with it all the silt and fine material.

The elevation of the bottom of the galleries is approximately 222.5 feet above mean sea level, or about 10.5 feet above the normal river level. An

examination of the records of the river elevations kept by the lock tender at Barge Canal Lock No. 8, about a quarter of a mile west of the pumping station, revealed the fact that the river had risen from an elevation of about 214 feet at noon on March 13 to 228 feet at midnight on that date, over one-half of the rise, 9 feet occurring between 2 and 3 o'clock. The river reached its maximum elevation of about 230 feet at 4 p.m. on March 14; and after that time the elevation gradually fell, reaching 224 feet on the 20th and 222 feet on the 21st of the month. The elevation of the water in the wells, figured from the record of the vacuum on the pump suctions, rose more slowly than that of the water in the river, the maximum rate of the rise being about 1 foot per hour. The river elevation was, therefore, for a considerable time, several feet above the elevation of the water in the wells. On the afternoon of the 13th this difference varied from 6 feet at noon to about 14½ feet at 3 p.m., dropping again to 7 feet at midnight. During the entire day of the 14th the elevation of the water in the wells, and from that date on gradually decreased, the difference on the 20th being only about 9 inches.

Compared with the elevation of the bottom of the pipe gallery, these figures indicate that the river surface was above the floor of the gallery from 3 o'clock on the afternoon of March 13 until March 20, and that during the afternoon of the 13th and during the 14th the river surface was between 6 and 8 feet above the bottom of the gallery while the water in the well remained below the bottom of the gallery.

Apparently, therefore, from the 13th to the 20th of March there was nothing to prevent the polluted water of the Mohawk River from flowing from the river to the galleries through the open 24-inch suction line, then along the galleries and, as indicated above, down through the wash holes a few feet from well No. 1, and through a few feet of coarse gravel into the well either by way of the joints in the stonework or up through the open bottom. The largest rate of flow into the well by this means would, of course, have occurred on the 13th and 14th of the month, when the difference in elevation between the surface of the river and the surface of the water in the well was greatest, and would have gradually decreased as the difference in elevation became less. This is in general accordance with the evidence as to the turbidity in the city water, which is said to have been the greatest on the night of the 13th and during the 14th, gradually becoming less during the week and finally disappearing about the 19th or 20th.

As a result of this investigation these open suction pipes to the river were removed. This experience is a most interesting one to illustrate the sensitiveness of a water supply to pollution, and particularly the extra precautions to be taken in spring flood times.

The question which is most commonly asked and discussed, however, is whether all the water in the wells comes from the river. In the minds of many the water filtered through from the river, since the river flows over the surface of the same bed of gravel as the wells. The river bottom is a layer of silt which is undoubtedly quite imper-

meable. However, that the level of the river affects the level of the water of the well cannot be denied. This was clearly demonstrated by the experience of 1918.

The Mohawk River at this location is part of the Barge Canal and is therefore maintained at a constant pool level of about 212 elevation. In December, 1918, the State opened the gates at Visscher's Ferry which lowered this pool level about 5.5 feet. The effect of this was to lower the water in the wells very materially and to cause serious alarm to the citizens of Schenectady. The closing of the gates brought the water back to its original level in the wells. It took three hours for the water in the river to reach its normal level after the closing of the gates and five hours more before the wells were normal.

To some this seemed confirmation of the contention that the water in the wells came from the river, but after some reflection it would be difficult to sustain this. Four studies have been quite convincing that water does not filter through the silt and thus become the source of this supply.

Relative elevations of river and well waters

First, the normal elevation of the wells when no pumping takes place is higher than the elevation of water in the river. This was demonstrated by shutting down pumps at 7 a.m., June 28, 1928 and taking readings eight hours after. The elevation of water in 3 wells averaged 213.35 while the river had an elevation of 212.56. These confirmed other readings made four weeks previously and show that natural flow must be toward the river.

The lowering of the river acts then to hold back the underground stream hydrostatically rather than to reduce the flow coming from it.

Temperatures

The second study pertained to the temperature of the water in the wells as compared to that in the river.

Readings taken in January and February showed the average of 7 readings to be

Well No. 1	43°
Well No. 2	51°
Well No. 3	52°

The average of 5 readings taken in July showed

Well No. 1	60°
Well No. 2	52°
Well No. 3	49°

Well No. 1 has a more variable temperature than No. 2 or 3 which is explained as due to the fact that No. 1 has open joints in the sides thus allowing water from upper courses of gravel more affected by surface climatic conditions to enter the well. The other two wells are open at the bottom only. At depths 40 to 60 feet below the surface the temperature of ground water from 40 to 50°F. geologists state that in this latitude the temperature at 55 feet depth remains constant. At the temperature found the water in wells 2 and 3 are remarkably constant, throughout the year, and must be from lower levels rather than surface. It would be difficult to demonstrate that the water from the Barge Canal pool heated by July sun could be cooled to a temperature of 49° in a distance of less than 400 feet through the ground. This would also seem improbable when it is taken into consideration that the experience of 1918 in regard to the influence of lowering river indicated that it took only eight hours for well and river to become hydrostatically normal.

Analytical data

The third study included analysis of the water to discover their corresponding characteristics.

The chemical analysis was most interesting. Samples of well No. 2 and Mohawk River at lock No. 8 a little above the water plant gave a comparison reported by Mr. Morris Cohn of the Sewage Disposal Plant Laboratory, as shown in table 1. Samples were taken June 1, 1928.

Mr. Cohn in commenting on the water stated:

Well Water, Well No. 2: The samples of water taken for chemical analysis were of the usual excellent physical appearance and characteristics. The water was colorless and, for all practical purposes, odorless. There was no turbidity and no suspended matter. The forms of nitrogen were present in quantities normal for this water. Chlorides were normal for the section. The water was of moderate hardness, the major portion of the total hardness being in the alkalinity form.

River Water—Lock No. 8, Barage Canal: The physical characteristics of the river water were normal for the stream. The color was a moderate yellow-brown. The odor, both hot and cold, was of a slight vegetable nature. The quantity of suspended solids was quite distinct. The quantity of free am-

monia was about twice that in the well water; the albuminoid ammonia content was somewhat more than double that contained in the latter. The quality of nitrites was markedly higher but the nitrates were but one-half of that in the well water. Chlorides were practically the same in both samples. The total hardness of the river water was about one-half that of the City supply and the alkalinity had the same ratio. It is evident that the water in the river contained appreciable organic pollution in the partially oxidized state. It is interesting to note that the difference in total hardness in the two samples is practically made up by the difference in the alkalinities or temporary hardnesses of the two waters. It would appear that the City supply had come in contact with ground minerals that had imparted a higher bicarbonate content than was the case with the surface water supply.

TABLE 1

CHARACTERISTIC	CITY WELL NO. 2	RIVER LOCK NO. 8
Color	Colorless	Yellow brown
Odor, hot	0	Vegetable
Odor, cold	0	Vegetable
Turbidity	0	0
Nitrogen as free ammonia	0.10	0.19
Albuminoid ammonia	0.10	0.24
Nitrites	0.003	0.040
Nitrates	0.847	0.410
Chlorides	5.5	5.0
Total hardness	118.8	61.0
Alkalinity	99.8	47.0

Results in parts per million.

At the same time the chemical analysis was made, samples were taken to the Ellis Hospital Laboratory for bacterial analysis. Dr. Kellert reported:

A. Well Water, Well No. 2, Rotterdam:
Bacteria per cubic centimeter, 2.
B. coli absent in 10 cc.

B. Well Water, Well No. 2, Rotterdam:
Bacteria per cubic centimeter, 2.
B. coli absent in 10 cc.

D. River Water, Lock 8:
Bacteria per cubic centimeter, 800.
Presumptive test for B. coli positive in 1 cc.

E. River Water, Lock 8:
Bacteria per cubic centimeter, 1200.
Presumptive test for B. coli positive in 0.1 cc.

A study of these shows great differences. However, most of this could be accounted for by filtering action of ground between the two places. There certainly can be no direct open connection, however.

Borings

The fourth study further to carry out the argument consisted in making a boring between the river and the wells. This was made a point 190 feet from the well and 60 feet from the river. The geologist reports in regard to this:

> The boring penetrated loam, coarse and fine sand to elevation 124.79 feet when hard pan (blue clay enclosing gravel stones) was struck. The blue clay continued, interrupted by one layer of coarse sand, to bedrock at 59.78 feet in sandstone. No free water was met with in this boring. The materials were wet, indicating that the pore spaces of the gravel and sand were filled with water, but no flow of water rising in the casing was met with until rock was penetrated 15 feet when water rose in the casing to elevation 216.54 feet. The normal water elevation of the river is 212 feet. This boring penetrated the porous sand and gravel deposits to the depth of 57.31 feet below the level of the river. The inference to be drawn from these data is that there is no free flow of water from the river to the wells.

A boring on the opposite of the well and some distance away revealed a freely flowing course of water in a stratum of coarse sand at elevation 203.16.

These would help to establish proof that water was directed toward rather than away from the river.

To assure further the safety of the purity of the supply, precautions are taken to chlorinate during high water and in recent years special attention is paid to prevent cross connections between the city supply and the river supply used by some of the industries. Chlorination by these users is required at all points of danger.

Schenectady enjoys one of the finest waters for a city of her size. The water comes through the distribution system remarkably cold throughout the whole year and is of excellent quality. The only criticism is the hardness which for some purposes creates a problem. This seems to have no harmful effect upon the health of the community, however, and it is hoped that no measures will be found necessary which will alter the pleasing qualities of the present supply.

OBSERVATIONS ON FILTER SAND SHRINKAGE[1]

By W. M. Wallace,[2] Roberts Hulbert[3] and Douglas Feben[4]

In the first part of this paper it is the intention to state briefly the extent to which some promising methods of filter sand maintenance, previously developed and applied on an experimental scale, have so far been found to give favorable results when adopted in actual plant practice. This part of the discussion will be confined to recent experience in Detroit with washing filters at even higher velocities than has been employed here in the past. The use of exceptionally high wash rates has only lately been made possible here by the expedient of cleaning the sand in all the large filters. It is possible now, on the basis of more than one year's trial of the maximum available wash on nine of the large filters, to indicate what the probable final answer may prove to be to such troublesome questions as these: (1) Is the highest available rate of wash here, which averages about 33 inches per minute rise (producing about 36 percent average sand expansion) going to prove sufficient to prevent re-coating of the sand? (2) Will such a rate prove high enough to prevent the formation of mud deposits or "shelves" along the filter walls on the gravel surface? (3) Is it going to prove sufficient to greatly minimize or prevent sand shrinkage? And, (4) although experimental findings indicated that a 50 percent, or thereabouts, sand expansion might be necessary to accomplish these ideal results, can they be obtained with less than a 50 percent expansion wash?

The latter part of this paper is concerned largely with the problem of sand shrinkage. Among the 80 filters in the Detroit plant a few have never exhibited this phenomenon to a noticeable extent, but, in contrast to these, others have shown moderate to very pronounced shrinkage of the sand away from the filter walls, resulting in cracks of various degrees in width. Hence the opportunity here to observe and study sand shrinkage has been unusually favorable, and the attempt has been made to take advantage of it. Naturally, the fac-

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tors that seemed to promote and aggravate the trouble were sought first. Once these were discovered and their importance recognized, it has not been difficult since then to find definite means of controlling shrinkage and to apply them.

In Detroit, then, this sort of filter ailment is no longer acute, but rather convalescent. A doctor naturally feels relieved to know his patient is recovering, but his interest in the disease per se, particularly his desire to know its cause, has not ceased on that account; on the contrary, it has doubtless become heightened by his contact with another case. So, in this instance of a troubling filter disease, it is understandable that here too there remains the keen desire to gain a better insight into the nature and origin of the primary forces which induce sand shrinkage. In the absence of rather definite ideas, or a particular theory to stimulate and form a basis for investigation, progress toward that end is apt to be slow and haphazard. Perhaps, then, it is not unfortunate that a new theory of sand shrinkage has lately been developed here; one which seems to explain some recent observations, and put in order certain loose-ends of established fact. This theory appears, furthermore, to offer a simple and plausible explanation of why a confined body of sand, through which water is flowing at a fixed rate, must undergo some initial shrinkage. Hoping that such a theory may prove of interest to the profession, it is given herewith, along with some evidence to substantiate it.

PART I. RESULTS FROM THE USE OF MAXIMUM WASH AFTER SAND CLEANING

1. *Indications from experimental plant studies.* Some rather unusual results were obtained at the Detroit experimental plant by using a velocity of backwash sufficient to obtain an average sand expansion of about 50 percent. Briefly, these results were as follows: (1) the entrained coagulum was driven out of the sand very rapidly and completely; (2) growth of coating on the sand was prevented permanently; (3) mud shelves no longer formed at the junction of the gravel surface and filter walls; and (4) shrinkage of the sand no longer took place. In the main plant a rate of wash somewhat higher than normal (averaging about 28 inches per minute vertical rise) was used prior to 1929, with rather unsatisfactory results, from the standpoint of maintaining clean filter beds. The sand became light with accumulated coating, mud shelves formed along the filter walls, and shrinkage in many of the beds was pronounced. Excessive loss of sand,

apparent when a higher velocity of wash was applied, was a strong argument against washing the filters with a wide open valve.

2. *Cleaning sand permits higher wash rates.* Largely as an experiment, the sand in one row of eight filters was cleaned by lye treatment and subsequent hose washing early in 1929. After cleaning it was then found that a full wash could be applied to these filters with almost no loss of sand; and for the past year this maximum wash has been continued with good results. That the higher wash rates appear to have been successful so far in preventing rapid accumula-

TABLE 1

FILTER NUMBER	VELOCITY OF WASH (INCHES PER MINUTE)		SAND EXPANSION* (PER CENT AT 50°F.)		SPECIFIC GRAVITY OF SAND† (AVERAGE OF TOP 6 INCHES)				
	Before April 1, 1929‡	After April 1, 1929§	Before April 1, 1929	After April 1, 1929	As of February 1, 1929, coated	As of April 1, 1929, cleaned	As of April 1, 1930, one year's service	Rate of change per annum: Past year	Rate of change per annum: 1924 to 1929
49	30.4	37.0	31.4	41.2	2.550	2.600	2.596	−0.004	−0.020
51	30.4	34.7	31.4	37.7	2.548	2.633	2.633	0	−0.020
53	29.5	35.1	30.0	38.5	2.547	2.608	2.612	+0.004	−0.020
55	30.5	33.8	31.5	36.5	2.550	2.612	2.608	−0.004	−0.020
57	28.8	32.6	29.0	34.7	2.560	2.616	2.620	+0.004	−0.018
59	27.7	32.5	27.3	34.6	2.558	2.609	2.613	+0.004	−0.019
61	28.5	31.7	28.5	33.3	2.548	2.612	2.602	−0.010	−0.020
63	28.6	32.5	28.6	34.6	2.549	2.608	2.603	−0.005	−0.020
Average...	29.2	33.7	29.5	36.4	2.553	2.612	2.611	−0.001	−0.019

* Interpolated for clean sand having a 30 percent size of 0.715 mm., a fair average for these filters. Actual expansion may vary slightly from these figures, due to some variation in the size and specific gravity of the sand.

† These determinations are accurate to approximately 0.005.

‡ Measured with wash water valve approximately 61 percent open.

§ Measured with wash water valve approximately 90 percent open.

tion of coating is shown by comparison of the specific gravities of these sands, before cleaning, immediately after cleaning and after one year on the increased velocity backwash. These data are compiled in table 1. In examination of the figures it should be remembered that the specific gravity of clean sand is 2.650, and since the coating substance which accumulates on the sand grains in this instance is lighter than the sand, the reduction in gravity of the latter is proportional to the amount of coating on it. Prior to cleaning

these filters the sand had been in use five years, and at the end of this time showed a reduction in gravity from 2.650 to 2.553 on the dry sand basis, or a total reduction of 0.097, which is at the rate of 0.02 per annum. Table 1 shows that in the past year, employing higher rates of wash, practically no accumulation of coating on the sand took place. The very slight average decrease in gravity of 0.001 indicated by the tabulation is insignificant. In the case of only one filter in the series does there appear to occur what could be considered a significant decrease of 0.01 in the year's time, but, it should be noted, this happened to the filter sand which measurement shows had the lowest velocity wash.

3. *Mud shelf formation retarded and shrinkage reduced.* As a result of preventing growth of coating on the sand, the increased wash used on these eight filters during the past year has had quite a marked effect in reducing the degree of shrinkage exhibited. The extent of the latter which is now apparent in any of them seems to be in direct relation to the status of mud shelf formations on the gravel along the filter walls, and appears not to be connected in any way with the slight differences in the percent of coating on the sands. It may be rightly inferred from this, that the maximum rates of wash obtainable, as shown in table 1, have proven insufficient to entirely prevent side wall mud accumulation. While not preventing such deposits entirely, the rate at which the mud accumulates has been greatly retarded. That fact was apparent even last summer, when the intensity of wash was least, due to the high water temperature; during this past winter no appreciable growth of the mud deposits, formed during the past summer and fall, took place. It may be argued that poor distribution of wash water is primarily to blame for the wall mud, rather than too low a velocity wash. To some extent this is true, but it is quite apparent here that the higher the velocity of wash used the less trouble of this sort develops.

Further evidence appears in table 1 that a high velocity of wash is important, as well as good distribution. For this row of eight filters it is to be noted that the velocities of wash (and resulting sand expansions) decrease gradually as the filter numbers increase (down the column) owing to the fact that the higher numbered filters are located the greatest distance away from the wash water storage tanks. It is these lower wash-rate filters that still disclose the most marked tendency to accumulate mud shelves against the walls, evidence that velocity of wash has much to do with the extent of this trouble.

The fact should be emphasized that the general improvement in the condition of these beds has been marked, as the result of cleaner sand and high velocity washing. Side wall cracks have been reduced fully 50 percent, and in some filters eliminated almost entirely. It is thought from the results so far apparent that the maximum wash, applied the year around, will prove sufficient to prevent coating of the sand in the large majority of filters. However, this maximum wash does not, by any means, give a constant 50 percent sand expansion, which, it is felt, now more than ever, is about the necessary intensity to entirely prevent any accumulation of wall mud and consequent shrinkage, particularly where poor distribution of wash is also a factor. This was the indication from the experimental filter results, and it appears to be well substantiated by these observations in the main plant.

4. *Filter beds composed of finer sands are kept clean.* It has not been possible with the large majority of filters to fully confirm the exceptionally good results obtained with a 50 percent sand expansion wash at the experimental plant, because with the general run of large filters it is not possible to attain any such high average expansion as this. Since the maximum velocity available for washing the Detroit filters is considerably higher than the customary rates provided elsewhere, the fact that less than 50 percent expansion of the sand results, cannot be attributed to low velocity of wash water. The blame for this mostly rests upon the unusually coarse sand with which 77 of the 80 filters are filled. As it happens, the three remaining filters were equipped some five years ago with three different kinds of filter sand, each considerably smaller in size and more uniform in grading than the regular sand. The record of these three filters over the five years they have been in service is pertinent to the present discussion. As it happens, they furnish additional important evidence that higher expansion washing does keep the sand cleaner, and thereby minimize the usual filter bed maintenance troubles.

By referring to table 2 it may be seen how three filters (nos. 70, 74 and 78) compared in March, 1928 (before cleaning) to the average of all others in the plant. A comparison of the sand sizes, the velocity of wash applied, and the resulting sand expansions are also given in this table.

It is apparent from these data that the three sands (nos. 70, 74 and 78) have all remained much cleaner than the average filter sand in the plant. None of these three filters at any time during their

five years of service, have ever developed side wall mud accumulations, nor exhibited more than very slight shrinkage. The higher expansion wash obtained appears to be the only obvious explanation of these facts.

Another bit of evidence in favor of higher expansion washing remains to be given. In filter no. 70 the sand was restored to a gravity of 2.640 during the spring of 1929 by lye treatment and hose washing. In addition, 6 inches of the bottom sand was removed from the bed, in order to lower the freeboard enough to permit a full 50 percent expansion wash without danger of losing sand. During the year past this filter has been washed at a velocity of 34 inches per minute rise, producing an average sand expansion of about 53 percent with wash water at 50°F. Tests of this sand now, after

TABLE 2

	Size of sand		Velocity of wash (inches per minute)	Expansion of sand (per cent at 50°)	Status of sand after 5 years		
	10 per cent finer than millimeters	30 per cent finer than millimeters			Specific gravity		Per cent coating
					As of March 1, 1930	Change in 5 years	As of March 1, 1930
Plant average..........	0.62	0.72	29.2	29.5	2.558	−0.092	12.8
Filter no. 70	0.56	0.61	28.6	43.5	2.620*	−0.030	4.0
Filter no. 74	0.58	0.63	28.4	41.0	2.610	−0.040	5.1
Filter no. 78	0.57	0.63	28.2	40.5	2.601	−0.049	6.5

* Gravity of sand no. 70 as of March 1, 1929, or after four years' use.

more than a year of such washing, show its gravity to be 2.643. If anything, the amount of coating on the sand has decreased; at least it may be said that the wash used has proved sufficient to prevent any accumulation of coating.

5. *Clean filter sands give longer runs.* Cleaning of all filter sands in the main plant was started in earnest last November and completed April 10 this year. The process used was that described in the earlier paper. The specific gravity of the coated sand before cleaning averaged 2.558 for seventy filters (average of top 6-inch sand layer), as compared to 2.610 after cleaning. The difference corresponds to a 60 percent reduction in coating, from the original 12.8 to 5.2 percent. The cost of the cleaning process was approximately $55.00 per filter of 1088 square feet sand area, or five cents

per square foot. The surface elevation of the sand was lowered about 2 inches by cleaning, which indicates that about 180 cubic feet of coating substance was removed per filter, equivalent to over 11 tons dry weight. Enough has been said to convince one that restoration of the filter sand by this process is decidedly worth while. One other important result, and one calculated to clinch the argument from the operator's viewpoint, is the fact that much longer runs are obtained with clean sand beds. In Detroit, a comparison made last December between 16 clean filters (Gallery 3) as against the remaining 64 dirty ones (Galleries 1, 2, 4 and 5) showed an average run of 33 hours clean, compared to 26.4 hours dirty, an increase of 6.6 hours or 25 percent. This increase in service period can be accounted for in part by the reduction of 0.75 foot in initial loss of head, and probably in part is attributable to deeper penetration of the floc into a clean sand.

PART II. SHRINKAGE THEORY AND OBSERVATIONS LEADING UP TO IT

An earlier paper[5] pointed out that pronounced shrinkage of sand from the side walls may usually be observed in filters containing (1) heavily coated sand, and (2) deposits of hard mud on the gravel along the side walls. Of these two factors concerned, the mud accumulation is the most important, since its presence greatly aggrevates the force which induces shrinkage of the bed, namely a flow of water directed more or less horizontally away from the filter wall toward the center of the filter, compressing the sand in the same direction during filtration. To demonstrate the existence of such a force, and to measure its extent, manometer tubes were placed in the sand bed, and readings of the water elevation in them taken during the filter run. The difference between the water surface elevation on the filter and the level to which the water rises in the manometer tube, is, of course, the loss of head in the sand at the depth to which the tube penetrates. This idea is not new, but was in fact used by J. H. Fuertes years ago in the Harrisburg negative head case, and more recently was recommended as a simple method of determining pressure distribution in the sand bed by Weston Gavett, in his excellent discussion of Wolman and Powell's paper on filter sand shrinkage.[6]

[5] This JOURNAL, November, 1929, page 1445.
[6] *Eng. News-Record*, December 2, 1920, 85: 23.

In order to procure accurate readings of the water height in the manometer tubes, the apparatus shown in figure 2 was devised here. It consists of an electrical circuit, comprising a six-volt battery, a galvanometer and electrode. The circuit is closed when the point of the electrode, being lowered into the tube, makes contact

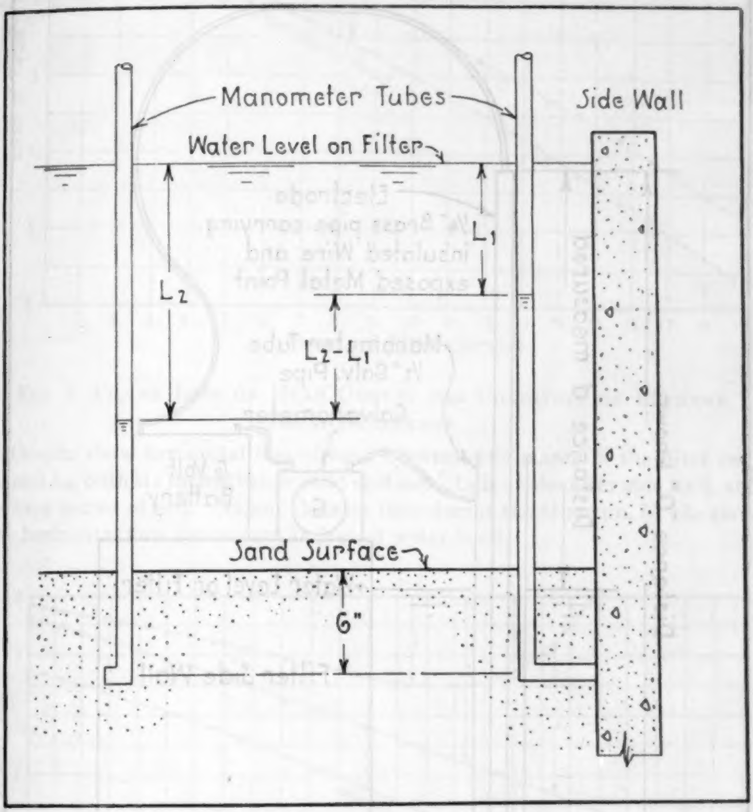

FIG. 1. LOCATION OF MANOMETER TUBES FOR MEASURING HORIZONTAL FLOW COMPONENT, L_2-L_1

with the water surface in the latter, which is indicated by deflection of the galvanometer needle. That the readings so taken are quite accurate is shown by the close plotting of the points along the curves of figures 3, 4 and 5.

Figure 1 shows the location of the manometer tubes in the sand bed. These were screened to prevent the entrance of sand and

accurately placed to the arbitrary depth of 6 inches beneath the sand surface used in these experiments. If L_1 represents the loss of head in the tube against the side wall, and L_2 that in the other, then L_2-L_1 is

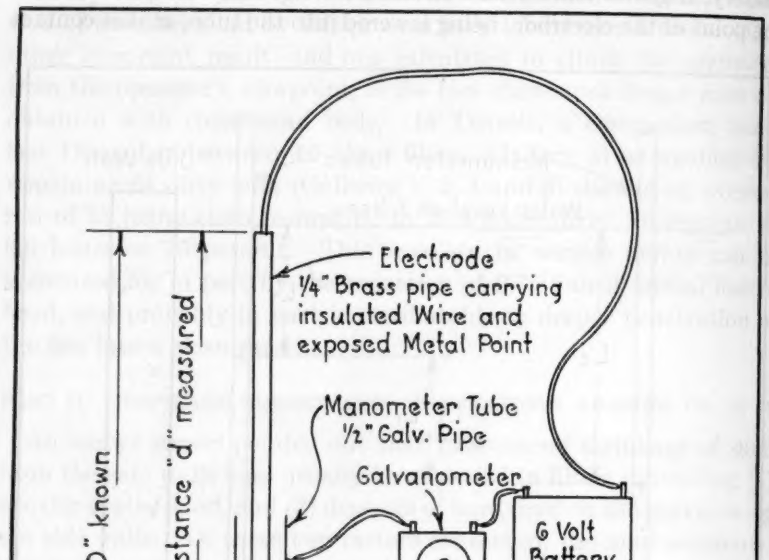

FIG. 2. ELECTRODE AND CIRCUIT FOR MEASURING LOSS OF HEAD, D–d

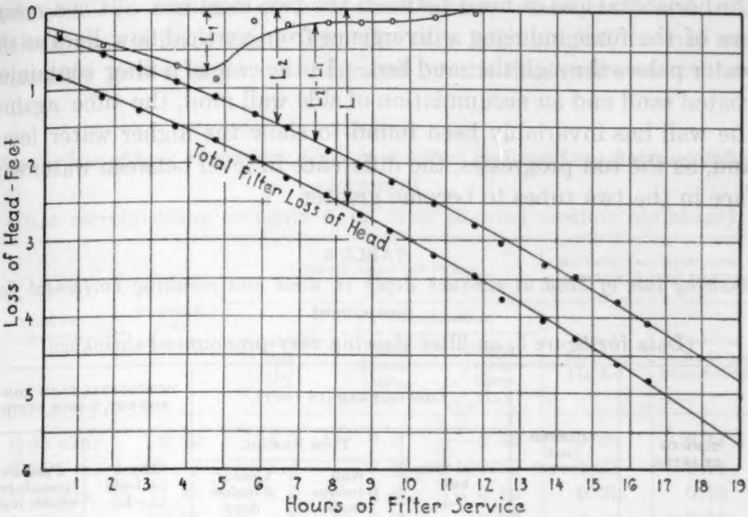

FIG. 3. FILTER LOSS OF HEAD CURVES FOR CONDITION OF EXTREME SAND SHRINKAGE

Graphs show horizontal loss of head between two points in the filter bed, L_1 and L_2, both six inches below sand surface. L_1 is at the filter side wall, and L_2 near center of bed. At any definite time during the filter run, L_2-L_1 gives the horizontal flow component in feet of water head.

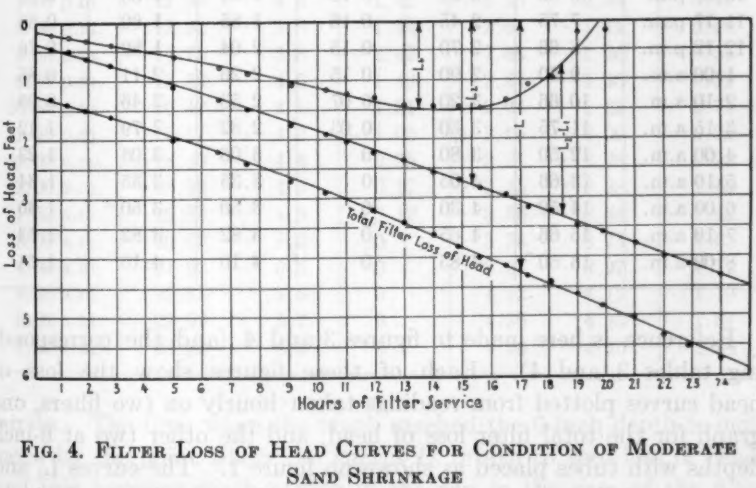

FIG. 4. FILTER LOSS OF HEAD CURVES FOR CONDITION OF MODERATE SAND SHRINKAGE

Graphs show horizontal loss of head between two points in the filter bed, L_1 and L_2, both six inches below sand surface. L_1 is at the filter side wall, and L_2 near center of bed. At any definite time during the filter run, L_2-L_1 gives the horizontal flow component in feet of water head.

the horizontal loss of head between the two locations. It is a measure of the force inducing a divergence from vertical flow lines as the water passes through the sand bed. In the case of a filter containing coated sand and an accumulation of side wall mud, the tube against the wall has invariably been found to show the higher water level, and, as the run progresses, the difference in level between water surface in the two tubes to become greater.

TABLE 3

Showing loss of head at six-inch depth in sand and resulting horizontal flow component

(Data for figure 3, on filter showing very pronounced shrinkage)

TIME OF READING	ELAPSED TIME (HOURS)	LOSS OF HEAD (IN FEET)			HORIZONTAL FLOW COMPONENT, 6-INCH DEPTH	
		Total filter loss (L)	Tube location: Wall, 6 inches deep (L_1)	Tube location: Center, 6 inches deep (L_2)	Head (in feet) (L_2–L_1)	Pressure (pounds per square inch)
3:55 p.m.	0.42	0.85	0.26	0.33	0.07	0.03
5:07 p.m.	1.66	1.10	0.44	0.56	0.12	0.05
6:09 p.m.	2.66	1.30	0.53	0.74	0.21	0.09
7:10 p.m.	3.66	1.50	0.73	0.92	0.19	0.08
8:14 p.m.	4.75	1.70	0.90	1.13	0.23	0.10
9:17 p.m.	5.75	1.95	0.14	1.35	1.21	0.50
10:08 p.m.	6.66	2.20	0.18	1.54	1.36	0.55
11:17 p.m.	7.75	2.45	0.16	1.85	1.69	0.68
12:12 p.m.	8.66	2.70	0.15	2.04	1.89	0.76
1:00 a.m.	9.50	2.90	0.15	2.26	2.11	0.85
2:10 a.m.	10.66	3.20	0.07	2.53	2.46	0.99
3:15 a.m.	11.75	3.50	0.03	2.82	2.79	1.12
4:00 a.m.	12.50	3.80	0	3.06	3.06	1.23
5:10 a.m.	13.66	4.05	0	3.35	3.35	1.34
6:00 a.m.	14.50	4.30	0	3.50	3.50	1.40
7:10 a.m.	15.66	4.65	0	3.82	3.82	1.53
8:00 a.m.	16.50	4.85	0	4.10	4.10	1.64

Reference is here made to figures 3 and 4 (and the corresponding tables 3 and 4). Each of these figures show the loss of head curves plotted from readings taken hourly on two filters, one graph for the total filter loss of head, and the other two at 6-inch depths with tubes placed as shown on figure 1. The curves L_1 and L_2 are especially to be noted. They do not show the degree of shrinkage taking place, but the head of water L_2–L_1 is the force tending to

induce it. In the case of both these filters, which contained mud shelves, such a force exists from the very beginning of the run and constantly grows larger, as indicated by the divergence of the two

TABLE 4

Showing loss of head at six-inch depth in sand bed and resulting horizontal flow component

(Data corresponding to figure 4, on filter showing medium shrinkage)

TIME OF READING	ELAPSED TIME (HOURS)	LOSS OF HEAD (IN FEET)			HORIZONTAL FLOW COMPONENT, 6-INCH DEPTH	
		Total filter loss (L)	Tube location			
			Wall, 6 inches deep (L_1)	Center, 6 inches deep (L_2)	Head (in feet) (L_2–L_1)	Pressure (pounds per square inch)
6:35 a.m.	0.50	1.3	0.20	0.46	0.26	0.11
7:15 a.m.	1.17	1.4	0.25	0.50	0.25	0.11
7:55 a.m.	1.85	1.5	0.31	0.62	0.31	0.13
8:50 a.m.	2.75	1.6	0.38	0.74	0.36	0.16
11:00 a.m.	4.92	2.0	0.58	1.10	0.52	0.23
12:35 p.m.	6.50	2.2	0.77	1.34	0.57	0.25
1:35 p.m.	7.50	2.4	0.87	1.50	0.63	0.27
3:05 p.m.	9.00	2.7	1.06	1.75	0.69	0.30
4:15 p.m.	10.20	2.9	1.15	1.91	0.76	0.33
5:00 p.m.	10.92	3.0	1.22	2.03	0.81	0.35
5:55 p.m.	11.83	3.2	1.29	2.22	0.93	0.40
7:00 p.m.	12.92	3.4	1.41	2.35	0.94	0.41
8:00 p.m.	13.92	3.6	1.44	2.55	1.11	0.48
8:50 p.m.	14.75	3.8	1.42	2.68	1.26	0.55
9:35 p.m.	15.50	4.0	1.49	2.84	1.35	0.59
10:30 p.m.	16.42	4.1	1.31	2.94	1.63	0.71
11:15 p.m.	17.17	4.3	1.06	3.15	2.09	0.91
12:05 a.m.	18.00	4.4	0.95	3.21	2.26	0.98
1:00 a.m.	18.92	4.6	0.17	3.45	3.28	1.42
1:55 a.m.	19.83	4.8	0.12	3.56	3.44	1.49
3:00 a.m.	20.92	5.0	0	3.75	3.75	1.63
4:00 a.m.	21.92	5.3	0	3.97	3.97	1.72
4:55 a.m.	22.83	5.5	0	4.12	4.12	1.79
6:00 a.m.	23.92	5.7	0	4.30	4.30	1.87
7:00 a.m.	24.92	6.0	0	4.46	4.46	1.94

curves. The time when the crack reached the 6-inch depth is indicated by the point where the curves L_1 return to zero loss of head, and this happens much earlier in the run in the case of the filter (fig. 3) which showed most pronounced shrinkage, in this case result-

ing in a crack two inches wide at the end of the run. Figure 4 is representative of a filter which showed a final crack ¾-inch wide. These data seem to clearly indicate a horizontal flow component away

TABLE 5

Showing loss of head at six-inch sand depth and resulting horizontal flow component

(Data for figure 5, on filter without mud shelf and sand clean)

TIME OF READING	ELAPSED TIME (HOURS)	LOSS OF HEAD (IN FEET)			HORIZONTAL FLOW COMPONENT, 6-INCH DEPTH	
		Total filter loss (L)	Tube location			
			Wall, 6 inches deep (L_1)	Center, 6 inches deep (L_2)	Head (in feet) (L_2-L_1)	Pressure (pounds per square inch)
1:00 p.m.	1.00	1.60	0.50	0.50	0	0
1:55 p.m.	1.92	1.75	0.65	0.65	0	0
3:00 p.m.	3.00	2.00	0.83	0.83	0	0
4:10 p.m.	4.17	2.25	1.10	1.09	0	0
5:12 p.m.	5.20	2.50	1.35	1.49	0.14	0.06
6:20 p.m.	6.33	2.75	1.54	1.53	0.01	0
7:15 p.m.	7.25	3.00	1.81	1.73	0	0
8:18 p.m.	8.30	3.25	2.00	1.98	0	0
9:00 p.m.	9.00	3.40	2.18	2.15	0	0
10:20 p.m.	10.33	3.80	2.50	2.52	0	0
11:17 p.m.	11.30	4.10	2.73	2.76	0.03	0
12:10 a.m.	12.17	4.30	3.00	2.99	0	0
1:10 a.m.	13.17	4.60	3.21	3.32	0.09	0.04
2.20 a.m.	14.33	5.00	3.55	3.60	0.05	0.02
3:20 a.m.	15.33	5.30	3.83	3.94	0.11	0.05
4:30 a.m.	16.50	5.70	4.15	4.26	0.11	0.05
5:25 a.m.	17.42	6.15	4.38	4.60	0.22	0.09
6:10 a.m.	18.17	6.40	4.55	4.90	0.35	0.15
6:55 a.m.	18.92	6.75	4.73	5.09	0.64	0.28
8:10 a.m.	20.17	7.30	4.67	5.61	0.94	0.41
9:05 a.m.	21.10	7.60	4.60	5.95*	1.35	0.59
10:00 a.m.	22.00	8.00	4.48	6.30*	1.82	0.79
10:30 a.m.	22.50	8.10	4.21	6.50*	2.29	1.00
11:05 a.m.	23.10	8.30	3.98	6.75*	2.77	1.20
12:00 a.m.	24.00	8.50	3.72	7.10*	3.38	1.47

* Water level below 6-inch sand depth; figures given are taken from extended curve.

from the walls toward the center of the bed, tending to compress the sand in that direction. If the sand is coated, shrinkage takes place more readily and to a greater degree.

During a filter run observations were taken of the amount of shrinkage[7] at a point along the filter wall, simultaneously with measurements of the horizontal pressure component at the same place. These data comprise table 6, and are shown graphically as figure 7. The close correspondence between the form and slope of these two

TABLE 6

Showing relationship between sand shrinkage (as measured by width of wall crack at sand surface) and horizontal pressure component, during filtration

Data corresponding to figure 7

ELAPSED TIME (HOURS)	LOSS OF HEAD (IN FEET) AT 6-INCH SAND DEPTH		HORIZONTAL FLOW COMPONENT		SAND SHRINKAGE (IN INCHES)
	Wall tube (L_1)	Center tube (L_2)	Head (in feet) (L_2-L_1)	Pressure (pounds per square inch)	
1.0	0.40	0.57	0.17	0.07	
2.0	0.55	0.80	0.25	0.11	
3.0	0.66	1.05	0.39	0.17	
4.0	0.75	1.30	0.55	0.24	
4.5					0.1+
5.0	0.80	1.60	0.80	0.35	
5.5					0.3−
6.0	0.60	1.92	1.32	0.57	0.3
7.0	0.25	2.25	2.00	0.87	
7.7	0.25				0.4−
8.0	0	2.60	2.60	1.10	
9.0	0	2.97	2.97	1.30	0.9
10.0	0	3.44	3.44	1.50	1.0
11.0	0	3.72	3.72	1.60	1.1−
12.0	0	4.02	4.02	1.74	1.1
13.0	0	4.20	4.20	1.82	1.1+

curves (fig. 7), one representing horizontal pressure and the other the amount of sand shrinkage, shows that as the pressure becomes greater, the degree of sand shrinkage increases in accordance. This sug-

[7] The width of the wall crack, due to shrinkage, was measured by submerging in the filter a hollow metal tube, long enough to reach from the observer's eye to within a few inches of the sand surface. The bottom of the tube is sealed with plain glass from which is suspended an illuminated scale. By lowering the tube until the scale is immediately above the sand surface, at right angles to and in contact with the filter wall, the width of the opening may be observed and measured by the scale.

gests quite strongly that shrinkage is a function of the horizontal pressure component.

In the case of a perfectly clean filter, no investigation had been made until recently to learn whether, during the normal operation of such a bed, there existed any force tending to induce shrinkage, similar to that found in the case of a dirty filter. It was assumed there did

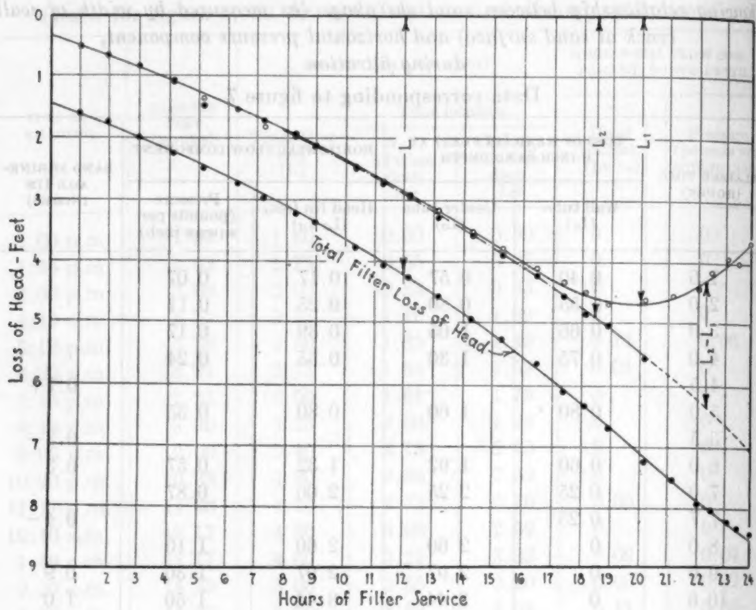

Fig. 5. Filter Loss of Head Curves for Condition of Very Slight Sand Shrinkage

Graphs show horizontal loss of head between two points in the filter bed, L_1 and L_2 both six inches below sand surface. L_1 is at the filter side wall, and L_2 near center of bed. At any definite time during the filter run, L_2-L_1 gives the horizontal flow component in feet of water head.

not, since it was firmly believed that the accumulation of side wall mud was entirely responsible. To the surprise of the investigators, when a similar experiment was made on a perfectly clean filter a short time ago, a slight and greatly delayed, but nevertheless measurable, increasing, horizontal loss of head was observed to occur between the same two localities in this filter, as well. The results of such a run on a clean sand bed are given in table 5, and graphically as

figure 5. In this case it is to be noted that no measurable difference in head was observed between the two tubes until late in the run. At the end of twenty-four hours the crack had not yet reached the 6-inch depth in the sand, and actually was only $\frac{3}{16}$ inch wide at the surface.

The three curves comprising figure 6 were obtained by plotting the horizontal pressures given in the last columns of tables 3, 4 and 5 against the corresponding hours of filter service. Here again, the degree of sand shrinkage exhibited by these different filters corresponds to the magnitude of the measured horizontal pressures.

Here then, in a clean filter, minus all the shrinkage factors previously postulated, existed the same force tending to induce shrinkage, that was thought would be found only in a mud-clogged filter.

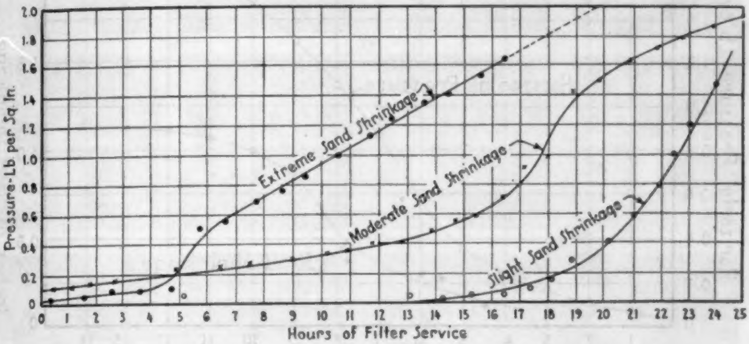

FIG. 6. HORIZONTAL FLOW COMPONENT IN POUNDS PER SQUARE INCH

What, one may ask, is the origin of this force, that induces a divergence from vertical flow in a clean sand bed. The following theory may appropriately be given here to explain the phenomenon.

When a filter sand subsides after a backwashing process, it is not conceivable that the size of void between the sand grains is 100 percent uniform throughout the entire bed. Consequently, when the filter is placed in service and water is flowing through the sand, its velocity through a void that happens to be smaller than those adjoining will be greater than the velocity of the water through those larger adjacent voids. This means that the pressure at this point will be less than the surrounding pressures, which tends to compress the sand in the direction of the smaller void. Now when the sand is moved in the direction of the lesser pressure the system does not tend

to attain equilibrium, rather its instability is increased, because the movement of the sand decreases the already smaller voids and increases the larger, and therefore increases the pressure differential which is moving the sand grains. Of course, when the void in any one particular spot in the filter becomes so great that the sand grains over it are not capable of self support, collapse takes place, and an entirely new set of strains are set up in that region of the filter. Following this reasoning throughout the entire filter, one can conceive of the small voids becoming yet smaller, the large voids becoming yet

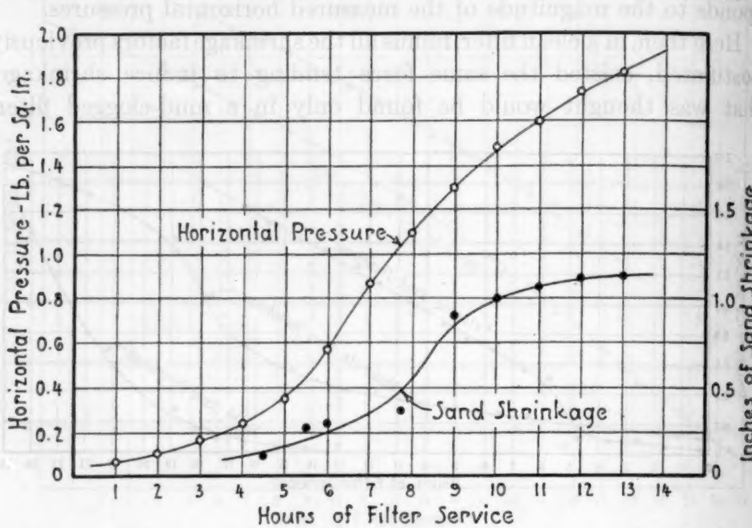

FIG. 7. RELATIONSHIP BETWEEN SAND SHRINKAGE (AS MEASURED BY WIDTH OF WALL CRACK AT SAND SURFACE), AND HORIZONTAL PRESSURE COMPONENT, DURING A FILTER RUN

larger, ultimate structural collapse of the large voids, and the entire sand bed shrinking in every direction. Such a movement of sand grains during filtration has been observed and photographed in a filter with glass sides. Figure 8 consists of four exposures each of four different sections of a sand bed. The photographs of each section were taken at regular time intervals of fifteen minutes or so apart. Each series may be seen to show clearly, that rearrangement of the sand grains, horizontally as well as vertically, to form a more compact filter, has actually taken place from one exposure to the next.

At this point it may be politic to strictly define what is meant by shrinkage, to obviate any possible misinterpretation. When a body

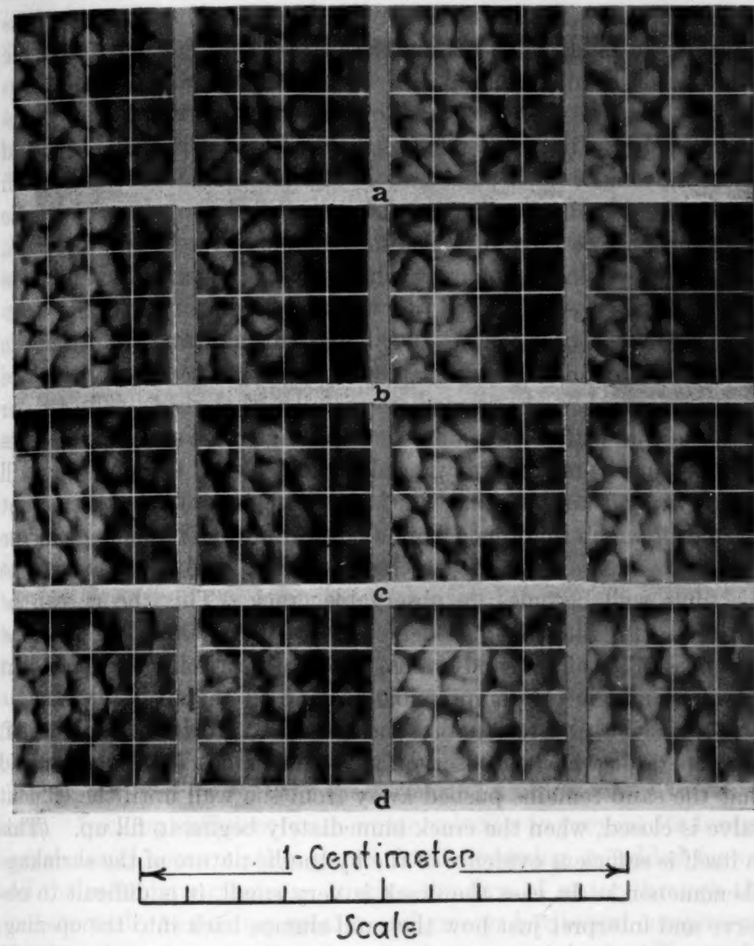

FIG. 8. HORIZONTAL AS WELL AS VERTICAL MOVEMENT OF SAND GRAINS DURING FILTRATION, RESULTING IN SHRINKAGE, OR COMPACTING OF THE SAND MASS

In each series: First section, 15 minutes; second section, 30 minutes; third section, 45 minutes; fourth section, 90 minutes.

is said to shrink it is understood that a volumetric contraction takes place, resulting in a movement of all the body surfaces toward its

center, a reduction of total surface area, and a closer grained internal structure. This is what is meant when the sand bed is said to shrink in every direction. In other recent discussions of this phenomenon actual sand shrinkage seems to have been confused with mere settlement of the sand. The writers of this paper do not agree that the two terms "shrinkage" and "settlement" should be taken as synonymous. It is true that settlement of the sand will bring about a reduction in volume, but settlement is a gravitational phenomenon, and so the movement takes place only in a gravitational direction, which is downwards; whereas the shrinkage described above is a hydraulic phenomenon, the force of which may be operative in any direction.

It is now pertinent to inquire what is taking place at the filter side walls. It has been explained why the sand has shrunk in every direction, consequently it follows that it has shrunk away from the side walls, doubtless a very slight amount, but still enough to make the amount of void in the immediate vicinity of the side-walls greater than the void within the sand bed. Here again the unequal velocities and pressures come into play, resulting in the sand being pushed still farther away from the wall. The latter being immovable, the amount of void of which it forms a boundary will always be more than the adjacent voids; and so the process continues until the shrinkage at the filter wall becomes an observable crack. This theory can be applied to any filter bed, the degree of shrinkage depending upon the compressibility of the sand grains, which in turn is largely dependent upon the amount of compressible coating on them.

It is possible now to visualize the origin and growth of a mud shelf. Any operator who has observed side-wall cracks will have noticed that the sand remains pushed away from the wall until the effluent valve is closed, when the crack immediately begins to fill up. (This in itself is sufficient evidence of the hydraulic nature of the shrinkage phenomenon.) In case the crack is very small, it is difficult to observe and interpret just how the sand slumps back into the opening, but if it be a large one, as a result of heavily coated sand, it will be noticed that one or more secondary cracks first appear, sometimes a foot or more from the wall, and the sand will then fall back suddenly, en masse, into the gap left by the shrinkage. In doing this it often sinks downwards several inches below the normal sand level. What is the interpretation of this?

It is a well known physical law that all bodies have a definite angle of repose. In the special application of this law to granular sub-

stances such as sand, it simply means that if a tube filled with sand be stood vertically on a plane surface, and the tube withdrawn, the sand will slump into a conical mound, its angle of repose being the angle formed by the side of this cone and the horizontal surface. This angle is a constant for that sample of sand. The angle of repose of clean sand is naturally smaller than that of coated sand, because the cementing, adherent characteristics of the coating helps to support the sand at a steeper angle. In the filter, the angle formed by the sand after shrinkage is greater than the normal angle of repose of that sand, so that when the hydraulic forces holding the sand away from the wall are removed, the cleanest sand at the bottom of the crack is the first to settle toward the wall, thereby removing some of the support from the sand above it. This process goes on from the bottom up, until the heavily coated sand at the top has room to move downwards as well as sideways, and then it suddenly settles bodily. In this way the fine, dirty sand from the surface of the filter starts its journey towards the gravel.

Unfortunately, as is often the case, there exists a lack of wash water outlets or strainers close to the borders of the filter, and this, in conjunction with the frictional resistance offered by the rough surface of the wall, and coupled with the large voids formed by gravel in contact with the wall (as compared with gravel in contact with gravel within the filter) all results in reduced wash water velocity at the very place where it is needed the most to remove accumulating mud. As a matter of fact, owing to the compactness of this mud shelf, and the tenacity with which it adheres to the gravel and side-wall, a much greater velocity of wash water should be provided here than elsewhere through the filter to prevent the formation and growth of this mud bank. Lacking sufficient wash water velocity to prevent its formation the mud accumulation grows, itself becoming an added factor, and the most important one to give the flow of water through the sand, normally vertical, a greater and greater horizontal component. This latter, in turn, induces more and more shrinkage. Hence the effects of the initial cause become added causes of the ultimate effect.

DISCUSSION

FRANK W. HERRING:[8] In discussing the paper presented last September, in which the results of the experimental work done in Detroit were described, Mr. Mansfield brought his comments to a close by

[8] Editorial Assistant, Engineering News-Record, Chicago, Ill.

expressing a hope that the results of observations upon the operation of the full size filters at Detroit would be published at a later date. Such a report has now been presented and the case for the Detroit method of operation has been greatly strengthened. On each point where it has been possible to check the experimental findings with observations on the full size plant, the former conclusions have been verified. Cleaning the sand of its encrusting substances has made possible the use of greatly increased washing velocities. The employing of the highest available velocities in Detroit has practically prevented the re-accumulation of coating material and has greatly retarded the formation of side wall cracks.

The higher net pressure observed close to the filter wall, and the attendant horizontal flow of water, in a bed of practically clean sand is explained in this paper in a very interesting fashion. The hypothesis presented appears sound with one major exception. It appears unlikely that a force of such small magnitude as that represented by the velocity head could be detected by the instruments used in the investigation. The Detroit rate of filtration, 160 m.g.d. per acre, is equivalent to a nominal velocity of 0.342 feet per minute, or if a porosity of 40 percent is assumed, to an actual velocity of 0.85 feet per minute. The velocity head corresponding to this rate is 0.000038 inch of water.

However, a study of the hydraulics of the sand bed discloses the existence of forces other than those mentioned in the paper and of undoubtedly greater magnitude than those due to difference in velocity head. The forces referred to are those due to differences in frictional resistance, or loss of head, between two neighboring pores of different size.

The law of capillary flow tells us that the loss of head per unit of length is proportional to $\frac{v}{A}$. If we assume that the flow is distributed uniformly over the area of the bed, it follows that discharge per unit area, or v, is constant. Under such conditions, then, loss of head is inversely proportional to the pore area. It follows that at any given depth the loss of head will be smaller in the larger pore, and the net pressure larger, bringing into existence a lateral difference in pressure tending to cause a flow of water from large pore to small.

The lateral pores operate to allow such flow, and tend to equalize the pressures in the two neighboring pores. For this pressure difference to be reduced, however, requires that there be an increase in

discharge through the large pore, toward the point where it would be proportional to the square of the pore area for equality in loss of head. Due to the frictional resistance of the lateral pores, a condition of equilibrium is attained before this limit is reached, and while there is a horizontal pressure difference still in effect.

The assumption made by the authors is that the discharge is equal for every pore in the bed. It is easy to see that an analysis based on their assumption leads to an even greater difference in loss of head than does that of uniform distribution over the area. It also becomes apparent that when equilibrium is established, the velocity in the large pores is greater than that in the small, so that any forces due to velocity head would be opposite in direction to those described by the authors.

The pressure difference here described operates to cause just such a lateral compacting of the filtering material as has been so ably described in the paper just presented. In addition the presence of the somewhat larger pores between the sand grains and the flat surface of the filter wall aggravates conditions at that point and no doubt provides the initial tendency for a crack to appear.

CHARLES R. COX:[9] One occasionally hears that the art of water filtration has reached the final stages of development. The fundamental experiments disclosed in this valuable paper indicate, however, that there are phases of the filtration of water which still need detailed scientific study for their elucidation. Unfortunately, these fundamental studies can only be made at experimental plants, such as those at Detroit, Cincinnati, Chicago, Cleveland, Ottawa, etc., because municipal filtration plants can not be manipulated experimentally as required in research work.

Literature upon filter sand shrinkage, mud ball formation, etc., indicates that there are several prevailing viewpoints upon this subject, in addition to the theory developed by the authors of the paper under discussion. One theory is that certain sands may have an inherent tendency to shrink because of the adsorptive properties of a colloidal coating upon the sand grains, or of the fine material which may be mixed with the sand. The use of clean sand of adequate size should prevent shrinkage, were poor quality of sand the cause of shrinkage. The second theory appears to be that shrinkage is due to the cementing action of organic matter, colloidal clay, etc., present in

[9] Assistant Sanitarian, State Department of Health, Albany, N. Y.

the raw water, and not properly coagulated and removed by the preliminary treatment.. Improved coagulation of the water, or more effective preliminary treatment would be the solution, if such be the cause of shrinkage. The third theory appears to be that ineffective washing of otherwise satisfactory sand beds is the fundamental cause of the shrinkage of the beds. Finally there is the theory developed in Detroit that shrinkage is due to the horizontal component of the hydraulic forces acting upon a sand bed. These theories will be considered in the light of experience at filtration plants in New York State.

Data relative to filter bed washing and sand shrinkage were furnished very kindly by those in charge of the filtration plants at Albany, Buffalo, Elmira, Ithaca, Newburgh, Olean, Poughkeepsie, Rensselaer, Waterford and Watertown, N. Y., and Greenwich, Conn. It is impossible to review the details of these data, but in general it appears that shrinkage is quite common. Thus serious shrinkage occurred at Poughkeepsie before the sand was replaced by other filter sand of more satisfactory size and quality, indicating the importance of the quality of sand, although it has been found that a slightly higher rate of wash, namely 27½ inches per minute, is very beneficial in the satisfactory maintenance of the present beds.

Sand bed shrinkage and the formation of mud balls has been quite serious at Watertown, where the raw water has an unusually high organic content incidental to swamp drainage, pulp and paper mill wastes. Improved washing alone has not been effective in cleaning the sand beds, and even caustic soda treatment of the sand did not remove the tough organic film which was deposited on the sand grains. The policy of frequent replacement of the sand has, therefore, been adopted at this plant.

The data from Albany and Ithaca, on the other hand, indicates that mud ball formation and clogged areas at various portions of the filter beds were due to the faulty coagulation of the water at periodic intervals, although poor washing is considered a factor at Ithaca and higher velocities of wash in summer appear desirable for the existing Albany filters.

Short filters runs and sand bed difficulties at Newburgh have been due to the periodic presence of large quantities of algae in the raw water. Prechlorination at this filtration plant has greatly improved filter operation and has prolonged the filter run. Previously to prechlorination at Newburgh, the beds were difficult to wash, because

the gelatinous film was firmly attached to the sand grains and could not be removed during the first portion of the washing period. Since prechlorination has been practiced, however, the film is different in character and is easily removed from the sand grains and decreased quantities of wash water are needed. This experience with the value of prechlorination indicates that the rate of wash of 24 inches in itself is not a controlling factor at the Newburgh plant, although the operator would favor a higher velocity of wash in summer.

The distribution of wash water is considered of vital importance at Buffalo and Waterford, because it has been observed at these plants that the washing of the edges and corners of the beds is defective, and that clogged areas occur at these points. The distribution and velocity of flow of wash water also is considered of great importance by those in charge of the filter plants at Albany, Greenwich, Newburgh and Poughkeepsie. On the other hand, the rate of rise of wash water at Elmira, Ithaca and Rensselaer is only 12 to 15 inches per minute, although the rate at Ithaca is considered inadequate when the raw water is very difficult to coagulate. The Elmira and Rensselaer filters, however, are of the circular tub type, fitted with mechanical agitators. Prechlorination is practiced at these latter plants. The indications are that both prechlorination and mechanical agitators are very effective in the prevention of filter bed clogging, serious sand shrinkage and mud ball formation, so that even low rates of wash water flow may be effective when accompanied by prechlorination and mechanical agitation. This is even true when the raw water has a high organic content such as that of the Hudson River water.

The sand bed at Olean shrinks ½ to ¾ inch during the normal filter run and small mud balls gradually accumulate in all portions of the beds. The rate of flow of wash water of 24 inches rise per minute is used. The use of a special piping, similar to that experimented with at Chicago, to apply additional wash water near the surface of the bed, is advocated by the chemist at this plant, rather than increased rate of flow through the main wash water system.

Several operators of other filtration plants in New York State have commented upon the absence of marked agitation when washing beds in summer at the conventional rate, and thus they look with favor upon the use of increased rates of wash water flow when the water is warm.

The above apparently conflicting data have been considered in the light of the information presented in the paper being discussed. It

is evident that certain of the experiences noted above may be explained by one or more of the theories of sand bed shrinkage enumerated above, and hence it may be concluded that no one theory adequately accounts for the phenomenon as a whole. A consideration of the magnitude of the forces of adsorption either inherent in some sands, or due to a film deposited on the sand grains, would lead one to conclude that their sphere of action is restricted to microscopic distances, and that they alone would not cause the shrinkage of sand beds. In other words, these forces do not appear of adequate magnitude to cause a movement of hundreds of pounds of sand for distances of ½ to 2 inches. In addition, we know of no instance where shrinkage has occurred in an idle bed, and hence in the absence of hydraulic forces. On the other hand, clean sand does not shrink to any extent, and the horizontal component of hydraulic forces, discussed in this paper, are only significant when the surface sand is partly clogged, and thus when the resistance to flow differs in various portions of filter beds. Therefore, these hydraulic forces and those of adsorption are concurrent, and related to a certain extent, in so far as sand bed shrinkage is concerned.

These horizontal components of the hydraulic forces are of adequate magnitude to account for sand bed shrinkage, following which the adsorptive surfaces would be brought close together and thus permit the adsorptive forces of a dirty sand, or one of improper quality, to maintain the sand in a compacted form, until the bed would be effectively washed. In other words, the theory developed at Detroit seems to account for the original shrinkage of a sand bed, and the forces of adsorption appear to account for the marked cohesion noted in beds having a tendency to shrink. Hence the two theories seem to harmonize. If this be the case, then the prevention of filter bed shrinkage requires the use of clean sand of proper size, the adequate preliminary treatment of the raw water, and effective washing of the filter beds.

Higher velocity of flow of wash water, when plant design permits, as advocated by the authors of this paper, undoubtedly will greatly improve the effectiveness of the washing of many filter beds, and also may overcome the effect of improper distribution of the wash water at the edges of the beds. The adjustment of the rate of flow of wash water to compensate for variations in the viscosity of the water with changes in temperature is logical, because the viscosity of water, and hence the degree of agitation per unit of flow, may vary from 50 to

100 percent with seasonal changes in raw water temperature. Thus any given rate of flow of wash water will not provide the same degree of agitation both winter and summer. The Detroit data seem to indicate that the conventional rate of 24 inches per minute is adequate with cold wash water, and, therefore, not adequate when the temperature of the water is over 50 to 60°F. A material increase in the conventional rate of flow of wash water, however, will not be feasible at many existing rapid sand filter plants, because of hydraulic limitation of the wash water system. It may be, however, that equally effective washing of the edges of these filter beds would result through the use of perforated pipes around the edges of the beds to discharge additional wash water at considerable velocity upward through the gravel and sand where the "mud shelf" and the more seriously clogged areas are usually located. This special wash water system would be used in conjunction with the conventional wash water system, through which the rate of flow of wash water would be increased in summer to about 30 inches vertical rise per minute, to compensate for lower viscosity of the water at such times.

In conclusion, it is hoped that these valuable studies will be continued at Detroit and elsewhere, and that it will be possible to extend them to include various qualities of raw water, size and character of sand, distribution of wash water through conventional strainer systems and special pipe grids around the edges of the filter beds, or elsewhere.

STUDIES IN WATER PURIFICATION PROCESSES AT CLEVELAND, OHIO[1]

By W. C. Lawrence[2]

Practically all the larger cities on the Great Lakes have comparable problems to solve in the clarification and purification of their raw water supplies. The general impression is that the Great Lakes waters are not troublesome, but such is not the case. The troubles confronting the operators of these plants are: periods of low turbidities, varying water temperatures over a wide range, high microscopic content in summer months, sudden changes in domestic and industrial pollution, and changeable wind directions and velocities. All of these factors affect the treatment of the water as regards clarification and sterilization.

In the clarification of these waters the processes consist chiefly of six steps in the following order: first, chemical dosing, which refers to the kind of coagulant used and the method of application; second, conditioning, that is the speed of mixing the solutions with the raw water; third, sedimentation, or time of detention; fourth, filtration; fifth and sixth, taste elimination and sterilization.

The treatment employed at both the Cleveland purification plants is comparable to most of the other plants treating Great Lakes waters.

DETENTION PERIODS

After the Baldwin Filtration Plant was placed into service in the fall of 1925, we had a fine opportunity to study detention periods required in the settling basins to get the best bacterial reductions, which had always caused us some doubt in operation.

Over a period of two years, 1926 and 1927, Baldwin Filtration Plant was operated monthly at rates of detention different from those maintained at the Division Filtration Plant. If a short detention period was used any particular month at one plant, a long period was used at the other plant. The detention periods ranged from 2 to 8 hours,

[1] Presented before the Central States Section meeting, September 26, 1930.

[2] Superintendent of Filtration, Cleveland, Ohio.

and the general conclusions drawn were that the density of the water was the important factor; that, when the water was at its greatest density about the first part of April, and again in the latter part of November, the coagulant required its longest settling period, and the best results were obtained with 8 hours' detention. In fact, from October to the end of March, 6 hours were required although the turbidity of our water is low, during these months, ranging for this survey from 2 to 50 p.p.m. on a monthly average. During the four summer months the detention periods required were shorter, May 4 hours, June $5\frac{1}{2}$ hours, and July and August 2 hours. The range of aluminum sulphate applied was from 0.8 to nearly 2.0 g.p.g., but the best results obtained were within the range of 0.8 to 1.1 g.p.g. This showed that increasing the amount of coagulant, above 1.1 g.p.g., had no value in B. coli reduction through the settling basins, and was only a waste of money. It was also noticed that with turbidities of less than 20 p.p.m. there was practically no noticeable difference between the mixing chamber effluent at Division Filtration Plant and the hydraulic jump effluent at Baldwin Filtration Plant, but with waters having turbidities of 20 to 50 p.p.m., the mixing chamber effluent showed better and required less time for settling than the hydraulic jump effluent.

Feeling fairly satisfied that the detention periods required for the different seasons of the year had been established, the next step was to improve the B. coli reduction through the sand beds.

With this thought in mind, experimental work was started on observing filter washings at various rates for water at different temperatures.

WASH WATER RATES

We are all aware that rapid sand filters get in a very bad condition by late summer, due chiefly to the accumulation of organic matter that hangs back in the washing of the beds. Then also, the microorganisms are abundant in the summer months, and become troublesome not only in shortening filter runs but by causing further accumulation on the sand grains. Filter cracks occur and piling up of sand around the edges of the filter beds then takes place. This condition produces bad effluent water, because around the edges of the filter boxes where the sand has pulled away, the water passes down very rapidly without filtering through the sand. Just inside the outer edge, however, where the sand and the organic matter have piled

up, the water to be filtered meets with a higher resistance and the rate of filtration is decreased considerably below the desired normal rate. With the normal rates of wash this condition persists, and throughout the summer becomes steadily worse and is not improved until late fall or early winter, as the organic matter decreases, and due to the change of density and viscosity of the water its lifting power increases, resulting in better washing.

Following the results obtained in the experimental work in the laboratory, with glass tube filters and also with one large filter unit in the plant, on varying the wash water rates with changes of water temperatures, it was decided to put this method of washing in practical operation at the Baldwin and Division Filtration Plants, January 1, 1929. At Division Filtration Plant where we expected to try a 40 percent sand expansion we found that we could only go to an approximate 30-inch rise per minute, due to the difficulty with the stems on the wash water valves, but at Baldwin Filtration Plant we were able to operate throughout the year at a 50 percent sand expansion and kept our sand beds in fine condition. Our operating program for adjusting the wash water valves at Baldwin Filtration Plant was fully described in a paper, by the writer presented at Central States Section meeting at Detroit in 1929.[3]

We continued this same schedule with a few slight changes this present year and have maintained all the filter beds at Baldwin in a very fine condition. This did not improve the bacterial efficiency through the filters to the degree we expected, but did increase filter runs about a third and saved about one-sixth of our wash water for the year, which has been an appreciable saving, as wash water is costly. However, improvements might be expected in the use of prechlorination treatment preceded by ammoniation especially during the summer months. This should reduce considerably the responsibility on operators of plants using water such as ours.

The writer has already made recommendations to the department heads to install an ammonia-chlorine treatment plant at Division Filtration Plant to treat the raw water and plans are now under consideration. If this works out successfully Baldwin Filtration Plant will be equipped for similar treatment.

TASTE AND ODOR PROBLEM

In August, 1927, the problem of tastes was again brought to the front following the appointment of a committee by the City Manager

[3] This JOURNAL, February, 1930, page 208.

for the purpose of studying the merits of Dr. Rudolf Adler's activated carbon process for the dechlorination and removal of chlorophenolic tastes from the filtered water supply of Cleveland. This process requires the application of chlorine in dosages sufficient to produce a minimum residual of 0.5 p.p.m. chlorine following a 30-minute detention period, during which all bacteria, organic and inorganic substances in the raw water are oxidized or destroyed. The residual chlorine remaining after the water has been treated in the described way is dechlorinated by passing water through a bed of activated carbon and transforming the free or active chlorine into the inactive chloride ion with great rapidity at the expense of carbon. Any chloro-organic or chlorophenol tastes that may result with strongly contaminated waters, and which are not destroyed by the dosage employed as specified, are removed by the carbon by adsorption and destroyed by the supposedly highly concentrated adsorbed layer of free or active chlorine.

Following the experiments by Dr. Adler, the committee recommended further experimentation by the City on a semi-plant scale.

The entire investigation covered a wide range of representative gas adsorbent and decolorizing carbons. Five different carbons were subjected to tests for loss of head, dechlorinating efficiencies, and the ability to remove chlorophenolic tastes.

Extensive studies were made at varying rates of flow of filtered water through different depths of carbon and with varying amounts of chlorine and phenol to establish the relative importance of these factors for the purpose of design and operation.

These tests were conducted over a period of a year and a half, involving some 6,000 samples, 19,000 tests consuming 16,000,000 gallons of filtered water with rates varying from 3,000 to 300,000 gallons daily at one to ten times sand filter rates, with carbon beds varying from 1 to 12 feet.

In general, none of the five activated carbons were able to entirely remove the chlorine applied to filtered water except at very low rates of flow. With increasing rates of flow and concentration of chlorine, the quantity of residual chlorine from the effluent was increased. None of the carbons removed chlorophenolic tastes from the chlorinated water at any rate of flow and with any depth of carbon when the amount of phenol present was equal to the maximum which has been found in the raw lake water intakes. However, when the amounts of phenol were less, one carbon with a 12-foot bed removed

the chlorophenolic taste at rapid sand filter rate of flow, and when flow was increased to three times the rate a decided chlorophenolic taste appeared in the effluent.

The cost of installing and operating the carbon filters under the conditions demonstrated in these tests was excessive and prohibitive. Whereupon the Committee reported the process impracticable for adaptation to the City's existing and future plant.

In view of the conclusions reached by this careful study, it was decided to continue with additional experiments on this problem of tastes. Superchlorination of the raw water and subsequent dechlorination of the filtered water by the carbon was suggested by the laboratory as an alternative for this process. As superchlorination has been practiced for a number of years, the only fact to be determined was the necessary dosages of chlorine required to destroy the amounts of phenol present at any one time in the raw water, and the depth of carbon as well as the rate of flow to remove the resulting excess chlorine. While tests indicated possible employment of smaller carbon beds, and higher permissible rates of flow, the amount of chlorine necessary would have to be increased at least seven times as much as used at the present time. However, the cost factor, though somewhat reduced, is still considerably higher, nearly double the cost of sulphur dioxide treatment. Furthermore, the life period, or duration of the carbon, is at the present time a matter of conjecture.

The carbon process being rejected, it was desirable to find a solution of the taste problem. The selection of the ammonia-chlorine process, due to costs and simplicity of installation and operation, was suggested and studied extensively by Mr. Braidech, Senior Chemist, and the writer.

Ammonia-chlorine process

This investigation covered a period of five months, during which 2400 samples were collected and some 9000 tests were completed. The results of the work conducted in the laboratory and confirmed by experimental plant tests were embodied in a report submitted to the committee as evidence that a successful solution of the problem has been found and that the following conclusions had been reached: first, that with pre-ammoniation taste and odors are prevented in as high a concentration as 1 p.p.m. phenol; second, that the process permits and retains higher residuals without producing chlorinous

odors and tastes; third, the process can be effected at nominal cost for installation and at low cost of operation and maintenance.

Following the recommendations of the committee, plants were equipped with ammonia installations and placed into operation in the late fall of 1929.

A city-wide survey of the distribution system was started shortly after the installation of the process, for a comparative study of sterilization efficiencies between chlorine and ammonia-chlorine treatment. This investigation was continued throughout the entire year of operation, involving about one thousand chemical and bacteriological samples. The data collected during the survey resulted in the following conclusions: chlorinous odors and tastes from residual chlorine have been prevented; no complaints have been received from the consumers due to the presence of chlorophenols; bacterial efficiencies have been greatly improved in the distribution system, and algae growths have been reduced to a minimum; cost of operation for the entire process did not exceed \$0.30 per million gallons of water treated.

A more detailed account of the earlier work was presented at the Ohio Conference on Water Purification in 1929 and is expected to be supplemented by another paper dealing with this and further investigations of the process at the same conference in 1930 by Mr. Braidech.

As a result of this series of systematic studies, the relative influences of these important and major unit processes have been evaluated to some extent.

CONCLUSIONS

First, the time required for settling of the treated water for the two plants has increased bacterial efficiencies; second, rise of wash water has improved filters and reduced consumption of wash water; third, the application of ammonia in conjunction with chlorination has prevented chlorophenolic and chloro-organic tastes, and has also improved sterilization.

At the present time, work is under way in connection with the relative merits of different coagulants and the various methods of mixing. This involves a comparative study of aluminum sulphate, chlorinated copperas, and ferric chloride, with regard to both the rate and type of mixing. In this work we are basing our comparisons mainly on two types of mixing,—rapid, as in the case of hydraulic

jump, and slow, as in the case of mixing chamber, over and under baffle. Results obtained so far, in a general way, indicate that slow mixing with either of the three coagulants produces better and more rapid sedimentation than is the case in rapid mix under identically the same conditions of varying turbidities. However, in either type of mixing, ferric chloride under same dosage and turbidity conditions required less time for settling. These studies have been made in the laboratory with waters under summer conditions, and further investigation on experimental plant scale is being contemplated with these different coagulants to determine more effectively the reduction of bacteria and microörganisms, and their respective effects on filter runs.

Thanks are due to Wallace & Tiernan Company, General Chemical Company, American Steel and Wire Company and Dow Chemical Company, for their advice and assistance.

Acknowledgments are due to Messrs. J. W. Ellms, Engineer of Water Purification and Sewage Disposal, Geo. D. Makepeace, Senior Bacteriologist, M. M. Braidech, Senior Chemist, and J. A. Marsh, Junior Chemist, for their coöperation and interest during this work.

SANITARY CONTROL OF RAILWAY WATER SUPPLY[1]

By R. E. Coughlan[2]

One of the duties becoming of more importance each year for the railroad water engineers is the control of the sanitary features of a railroad's water supply.

The federal authorities are maintaining close supervision over the water supplied for drinking purposes to passengers on the trains. In this regard each state has certain requirements with which it is necessary for all railroads to comply when operating within the boundaries of such states.

Although certain of the requirements of some states may conflict at times with those of other states, the railroads are anxious to comply with all requirements which will safeguard the health of the passengers transported each day over the vast network of the American railroads.

The representative engineering society of the railroads, the American Railway Engineering Association, has a special committee known as the Committee of Water Service and Sanitation. This Committee is in close coöperation with the Joint Committee of Railway Sanitation, composed of representatives from the Medical and Surgical Section, the Mechanical Division and the Engineering Section of the American Railway Association, and also the Public Health Service.

This Joint Committee was originally appointed to study the sanitary features pertaining to drinking water supplies, but recently its duties were enlarged to include the scope of general sanitary practices covering type, facilities and method of furnishing drinking water to passenger cars and other allied features of railway sanitation, as well as quality of the supply.

A manual of recommended practices is being developed in order to place before all of the railroads the approved recommendations to avoid possible arbitrary rulings on these matters by federal, state or local health authorities.

[1] Presented before the Illinois Section meeting, April 24, 1930.

[2] Supervisor of Water Supply, Chicago and Northwestern Railway Company, Chicago, Ill.

In the matter of providing safe water on passenger coaches, it is not only necessary to know that the original supply is safe and approved by the health authorities, but extreme care must be taken to know that the water is not contaminated in transferring it from the supply to the cars. The railroads are endeavoring to prevent the possibility of furnishing unfit water on their passenger cars and some of the practices aiming toward this end are noted below.

SANITARY PRACTICE IN SUPPLYING WATER

The hydrants should be placed in a pit, or box, the top of which is flush with the platform or surface of the ground. Quick opening type of valves should be used, and hose connections which permit the hose to be quickly attached and removed should be used. They should be self-draining, non-freezing, or adequately frost-proofed. Adequate drainage should be provided to carry away surface waste, as well as the water drained from the hydrants.

The hose for the delivery of water to the cars should be used for that purpose only. It should be thoroughly flushed out before each use. When not in use it should not be permitted to lie on the ground, unless the nozzle is protected from contamination, but should be stored in a suitable receptacle. In moving the hose from one locality to the other the workman should carry it so that the ends will not be dragging on the ground. Extreme care should be exercised to prevent the drinking water hose from being used for washing cars, toilets or other equipment.

The hose should have a smooth nozzle to minimize the adherence of dirt. Special devices to prevent the nozzle from touching the ground should, also, be used.

The inlet connections on the cars should be so located that there is no possibility of contamination from waste discharged from the passenger car.

The men, employed in the duty of watering cars, must be rigidly instructed. Personal cleanliness, as well as freedom from the possibility of being a carrier of typhoid fever or other diseases, should be required.

Where buckets are employed, these buckets should have spouts and complete covers, and should be used only for the purpose of watering cars. Special lockers, or boxes, should be provided for the sole purpose of storing these buckets when not in use. It is, also, desirable to keep these buckets in a separate compartment from the

ice when a cart or truck is used for the convenient transportation of such equipment from car to car.

It is essential that all supervisor officers be thoroughly informed as to the sanitary methods of handling the water supplied to cars, as the general rank and file of coach yard employes are not in a position to know the most sanitary methods to be employed.

As the watering of cars en route is usually done under the handicap of as short a space of time as possible, it is important that the foreman in charge of such work be ever alert to prevent any unsanitary method from being employed.

The necessity of providing safe drinking water for patrons, as well as employes of railroads at all times is not debatable.

During the flood conditions of the Mississippi Valley in 1927, the railroad water engineers, as well as all other available workmen experienced in sanitation, handled a condition of disease prevention which can be pointed to with pride. With some 18,000 square miles inundated, and the flood covering the dug and driven wells, with the pumping plants out of commission and the unsubmerged portions of the land crowded with human beings as well as stock, a system of chlorination was installed which undoubtedly saved many hundreds of lives.

Very few of the railroads chlorinate the water supplied to patrons, as a supply which has been already accepted by state or federal authorities is generally utilized.

When installing its own supply from wells, the customary precautions are observed to keep the water free from contamination, chlorine being the main preventative control of contamination on such work.

While the sanitary features of water supply represent the main features of railway sanitation, a broad definition of this sanitation will include refuse disposal, sanitary regulations of dining cars, boarding cars, hotels and restaurants.

The railroads recognize their responsibility in the matter of safe water supply, and although this work is still of comparatively recent origin, as pertaining to the phase of the water service engineer, it is a duty which has been gladly assumed by these employes and rapid progress may be expected each year in this important field.

THE IRON REMOVAL PLANT AT KOKOMO, INDIANA[1]

By F. P. Stradling[2]

The water supply of the City of Kokomo is taken from eleven deep wells varying in size from 6 to 16 inches. Eight of the wells are connected through 6-, 10-, 12- and 16-inch lines to a 20-inch suction line which discharges into a receiving well below the surface of the water. The water from these eight wells flows into the receiving well by means of a siphon action with a vacuum maintained upon the line at all times, varying from 9 to 24 inches depending upon the demand. The vacuum is maintained by a vacuum pump operated for a period of approximately thirty minutes three times a day, and then oftener, if the demand is above normal.

Two additional wells located about two-thirds of a mile east of the pumping station and equipped with Hill Tripp deep well pumps were installed in 1917. These wells are 16 inches in diameter with a 10-inch drop pipe and an 8-inch discharge. Each of the pumps is equipped with a 50 HP, two phase, Lincoln motor with three-stage Cutler-Hammer starting equipment operated by remote control at the pumping station. Prior to the installation of the equipment for the iron removal plant, these two wells discharged into the receiving well through a 16-inch cast iron line.

The additional well is a flowing well in the bottom of the receiving basin. The receiving basin is 15 feet in diameter and 50 feet deep.

The water from the wells contains iron in solution with an average content of approximately 3 p.p.m. This is sufficient iron to be troublesome in the distribution system and cause discoloration when a part of the iron becomes oxidized. In the new iron removal plant the water is first pumped from the receiving basin and discharged over an aerator. A motor driven deep well pump of 3 million gallons capacity is used for this purpose. The unit is housed in a building constructed over the receiving well. The pump is driven by a 50 HP Westinghouse 230 volt D.C. motor. The current for this

[1] Presented before the Indiana Section meeting, February 26, 1931.

[2] Superintendent, Kokomo Water Works Company, Kokomo, Ind.

motor is generated by means of a three-cylinder Diesel engine generating set. The engine was built by the Fulton Iron Works of St. Louis and all of the electrical equipment, including generator and starting equipment, was manufactured by Westinghouse. The motor is a variable speed motor ranging from 850 to 1150 r.p.m. which enables us to have almost perfect control of the water delivered to the aerator.

The discharge from the low service pump is connected with proper valves and check valves to the discharge from the two electrical wells previously mentioned. This line carries the water to the aerator and the water may come from either the receiving basin or from the electric driven wells as desired. The low service standby equipment is a 2.5 million gallon Worthington triple expansion pumping unit which can be used for either high or low service. A low service discharge from this pump connects to the 16-inch line to the aerator.

A Bailey flow meter registers the water passing to the aerator. The recording instrument being located in the pumping station adjacent to the starting box and control of the electric driven low service pump.

The aerator consists of four series of coke trays with five trays in each set. The water is discharged over the top of the trays by means of a perforated pipe distributor system. The water then flows down through the trays and is collected in a steel tank. The tank also provides the support for the aerator. The steel trays are approximately 5 by 2 feet, and the bottom of each is perforated with two hundred and ninety-six $\frac{1}{16}$-inch holes. The trays contain the coke in order to completely film out the water and obtain the removal of the CO_2 and hydrogen sulphide present which holds the iron in solution in the water. The space occupied by the aerator and tank is approximately 12 by 16 feet.

A louver constructed of wood completely surrounds the aerator to prevent the spray of water on the trays being carried outside the tank by wind currents. A line is provided so that the aerator may be bypassed if necessary.

From the tank the water flows to the filters through a 20-inch influent line. In the filter building we have 6 three-quarter million gallon concrete rapid sand filters. Each filter is equipped with Simplex loss of head and rate of flow gauges for convenience in telling when the filter should be washed and to give the rate of flow passing through the filter bed. Each filter has a sand area of 176 square feet. The

filter bed consists of Red Wing sand 30 inches in depth and supported on a 15-inch layer of graded gravel. The gravel was obtained from Cape May, N. J. The filters are constructed of concrete throughout and are of the false bottom type of rapid sand filter. The filter strainers are of the umbrella type and are placed on 6-inch centers over the false bottoms. The filters discharge into a clear water basin with a capacity of 120,000 gallons.

The filter wash water is supplied by 50,000-gallon elevated steel tank, 40 feet high. The wash water at the present time is averaging from 1.5 to 2.5 percent of the total pumpage. The length of filter run between washing is about 30 hours. The average pumpage on high service is 2 m.g.d. The filtered water is sterilized, using chlorine, which is applied by means of vacuum type chlorinators in duplicate.

The high service pumps take suction from the clear water basin. A storage reservoir of 750,000 gallons capacity operates in connection with the clear water basin on a float valve.

The total capacity of the nine suction wells is 3 m.g.d. and the capacity of the two electric wells is 2 m.g.d. In a recent survey by the National Board of Fire Underwriters we were given credit for a ten-hour run at a 7 m.g.d. rate which was based on the 5 million gallon well supply and using the available storage in connection with it.

Daily bacteriological tests are made on samples collected from the supply in the water company laboratory. Regular determinations for iron are also made in order to know that the iron removal plant is functioning properly. Complete removal of the iron from the water has been maintained since the plant was placed in service and we find that our consumers are very much pleased with the improvement in the quality of the water which the removal of the iron affords.

The entire plant consisting of aerating system, filter plant, clear water basin, wash water tank, necessary pipe lines and sewer cost \$56,000.00. The low service pumping equipment and housing cost \$19,400.00, making a total expenditure of \$75,400.00. The installation of the iron removal plant made it necessary to install the low service pumping equipment and, therefore, the cost of this equipment should be included in the total cost. The total cost is at the rate of approximately \$17,000 per million gallons capacity.

The plant has not yet been in service long enough to have figures on the cost of operation. An estimate on the annual cost of operation

consisting of low service pumping, labor, maintenance, depreciation, interest on investment, taxes and wash water is $12,000.00 per year. On the basis of last year's pumpage the estimated operating cost is at the rate of approximately $16.00 per million gallons or .016 cents per thousand gallons.

REPORT OF SECTIONAL COMMITTEE ON SPECIFICATIONS FOR CAST IRON PIPE, 1930[1]

(*Functioning under the American Standards Association*)

During 1930 work was completed on programs so far adopted for tests on cast iron pipe and fittings at three universities as follows:

At the University of Illinois under the direction of Professor M. L. Enger strength tests were made on 6- and 20-inch pit cast pipe and on 6- and 12-inch tees, crosses, quarter bends and wyes. These tests have added valuable and needed knowledge of the strength properties of pipe and fittings and in the case of the latter are showing how slight changes in the dimensions at important points will contribute a material advantage in strength.

At Iowa State College under the direction of Mr. W. J. Schlick, Drainage Engineer, trench load tests were completed on 20-inch pit cast pipe. Some of these tests are made with no internal water pressure and others with internal water pressure running up to about 1400 pounds per square inch. The reports on these tests offer a method of determining the allowable supporting strength of cast iron pipe subjected to internal water pressure and present average strength ratios for determining the supporting strengths with different pipe-laying conditions.

At Cornell University under the direction of Professor E. W. Schoder tests were completed on friction loss through 6- and 12-inch eighth bends, quarter bends, tees and crosses. These tests have been made with entry velocities varying from 3 to 10 feet per second and the fittings tested have been of two kinds, American Water Works Association standard fittings and others of shorter dimensions which the committee has under consideration. These tests are bringing out the effects on friction loss of the radius of curvature of the inside corners of tees and crosses as this inside radius is 6 inches in the case of American Water Works Association fittings and 2.5 inches in the short fittings. Some other fittings have also been tested

[1] Previous reports of this Sectional Committee appeared in the JOURNAL May, 1929, page 650; May, 1930, page 649.

with 1-inch radius at these inside corners and still others with sharp intersections such as are found in fittings used for steam.

Tests on organic coatings and linings were made at the shops of the American Cast Iron Pipe Company at Birmingham, Ala., under the direction of Mr. S. R. Church, Chairman of Sub-Committee 3-B on Organic Coatings.

Tests in service of cement lined pipes were inaugurated at the works of the Birmingham Water Company, Birmingham, Ala., by Mr. E. O. Sweet, Superintendent of Water of that Company and Chairman of Sub-Committee 3-C on Inorganic Coatings and Linings.

Sub-Committee 3-C, E. O. Sweet, Chairman, also adopted tentative specifications for Cement Linings for Cast Iron Pipe and forwarded them to Technical Committee 3 on Corrosion and Coatings for its consideration.

The comprehensive program of tests of various kinds being carried through by this Sectional Committee results in a great deal of new information on the properties of pipe and fittings, upon which revisions of existing specifications will be based, and all of which is now being carefully studied and analyzed by the three main Technical Committees and the fourteen sub-committees handling the various topics, comprising a total membership of 100 men actively engaged on this work.

SPONSOR SOCIETIES:

American Gas Association.
American Society For Testing Materials.
American Water Works Association.
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ABSTRACTS OF WATER WORKS LITERATURE[1]

FRANK HANNAN

Key: American Journal of Public Health, 12: 1, 16, January, 1922. The figure 12 refers to the volume, 1 to the number of the issue, and 16 to the page of the Journal.

Some Aspects of Corrosion in Cast Iron Main Pipes. J. R. BRADSHAW. Gas J., 186: 593–6, 1929. From Chem. Abst., 23: 4661, October 10, 1929. Eight samples of cast iron pipe of varying sizes, ages, and surroundings, some gas pipes and some water pipes, were studied for corrosion effects. Outside corrosion consisted essentially of ferrous oxide, etc., typical of electrolytic soil action. The internal deposits were essentially brown hydrated ferric oxide with various amounts of ferrous oxide. Pipe buried in firm clay stands up much longer than in a mixture of clay and ashes or in wet clay. A standard and unvarying quality of iron of high density should last almost indefinitely.—*R. E. Thompson.*

Quality of the Surface Waters of North Carolina. E. E. RANDOLPH. J. Elisha Mitchell Sci. Soc., 44: 70–4, 1928. From Chem. Abst., 23: 4757, October 10, 1929. Complete industrial analyses were made of about 200 samples of surface water collected from all the large streams of the state. The average amounts of dissolved total solids in the samples from the mountain, piedmont, and coast plain sections were 25, 55, and 60 p.p.m. respectively. The waters of the entire state are slightly alkaline and unusually soft. The average calcium content is from 4 to 6 p.p.m., and the average magnesium content, from 1 to 3 p.p.m. The iron content is very low.—*R. E. Thompson.*

Influence of Organic Matter and Its Removal in the Determination of Iron in Water. G. NACHTIGALL and M. BAYER. Arch. Hyg., 100: 35–9, 1928; Wasser u. Abwasser, 25: 305. From Chem. Abst., 23: 4758, October 10, 1929. In the thiocyanate method of iron determination, a clear solution and iron in the tervalent condition are necessary. In clear waters, bromine, hydrogen peroxide, and potassium chlorate are all satisfactory oxidizing agents, but the latter is preferred in waters containing much organic matter. The use of 1 to 5 drops of hydrogen peroxide gave favorable results, but larger quantities decomposed the thiocyanate pigment.—*R. E. Thompson.*

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[1] Vacancies on the abstracting staff occur from time to time. Members desirous of coöperating in this work are earnestly requested to communicate with the chief abstractor, Frank Hannan, 285 Willow Avenue, Toronto 8, Ontario, Canada.

Sixty-Fifth Annual Report on Alkali, etc., Works in England and Wales. T. Lewis Bailey. Ann. Rept. Alkali, etc., Works Proc. for 1928, 3–26, 1929. From Chem. Abst., 23: 4753, October 10, 1929. In connection with ammonia plant effluent disposal the following points deserve attention: (1) devil-liquor, containing most of the phenol, can be evaporated by direct heating in a hot chimney, evaporated by direct heating if the vapor can be taken to a steam boiler chimney, or some may be removed at producer fires; (2) steps can be taken to maintain a properly regulated flow of spent liquor from the works; and (3) as complete separation as possible of liquor from tar should always be effected at once. A section of the appendix gives an outline of progress on the analysis and constitution of ammoniacal and spent liquors, particularly in regard to the difference between actual oxygen absorption and the sum of the effects due to known constituents.—*R. E. Thompson.*

Standard Methods of Water Analysis. Progress Report of Sub-Committee No. 8 on Standardization of Water Analysis. (Tentative Method for Determination of Dissolved Oxygen in Boiler Feedwater.) Harold Farmer. Fuels and Steam Power (A.S.M.E. Trans.), 51: 12, 90–3, 1929; cf. C.A., 22: 2630. From Chem. Abst., 23: 4758, October 10, 1929. A tentative method for trial and criticism by the various industries is presented. Manganese chloride is used instead of manganese sulfate. The chemical reactions involved are given and the apparatus used is illustrated. Sufficient construction details are shown for making the cooling coil whereby water supplied in excess of 125°F., and above atmospheric pressure may be cooled to room temperature.—*R. E. Thompson.*

Oligodynamic Water Purification by Means of Catalytic Silver. G. A. Krause. Gesundh. Ing., 52: 500–5, 1929. From Chem. Abst., 23: 4985, October 20, 1929. Activated silver (Katadyn) kills all known pathogens. Finely divided silver is activated by metals below it in the electromotive series, such as palladium and gold. Sterilization is effected by passing the water over the Katadyn. —*R. E. Thompson.*

A Study of the Taste that Develops When Water Containing Phenol is Treated with Chlorine. F. Diénert and F. Wandenbulcke. Ann. hyg. publ. ind. sociale 1929, 298–301; Rev. hyg. méd. prev., 51: 489–92, 1929. From Chem. Abst., 23: 4758, October 10, 1929. In water containing phenol in concentrations higher than 10^{-10}, chlorine produces a taste like iodoform that is attributed to the formation of an oxychlorophenol. Quantitative experiments on both spring water and Seine water show that with increasing amounts of chlorine the degree of taste increases to a maximum and then decreases until no taste is present. The less phenol there is present, however, the higher is the chlorine concentration necessary to reach the maximum of taste, so that the intensity of taste is a function of the concentration of chlorine as well as of the phenol. The taste always disappears gradually in course of time; the slower the taste disappears, the stronger the taste was originally. If the free chlorine is destroyed by hyposulfite the taste disappears much sooner than without this treatment. However, even if hyposulfite is added before

the chlorine, some taste will appear and the safest way of preventing the formation of the unpleasant taste is destruction of the phenol by means of permanganate. When phenol gets into the water after the chlorine and hyposulfite treatments no taste is formed. These treatments, therefore, are effective and unobjectionable when, as is frequently the case, the only source of phenol is the tar material used in coating the pipe system through which the water circulates after leaving the plant. On the pure compounds, *o*- and *p*-chlorophenol and trichlorophenol, the observation was made that the gradual disappearance of the iodoform taste is due to the decomposition of the chlorine compounds. *o*-Chlorophenol gives the strongest and most persistent taste.—*R. E. Thompson.*

The Influence of Temperature Upon the Chlorine in Water. G. W. Schmidt. Arch. Hyg., 101: 290–6, 1929. From Chem. Abst., 23: 4985, October 20, 1929. The ability of distilled water, tap water, solutions of organic matter, and sewage to bind chlorine increases with the temperature.—*R. E. Thompson.*

Antiliton. M. S. Mileant. Ukrainskii Khem. Zhur., 4: 1, Tech. Pt., 23–9, 1929. From Chem. Abst., 23: 4758, October 10, 1929. Antiliton, a boiler compound, contains 20 percent of solids and 12 percent of tannins. When a water having 8° of temporary hardness and 16° of permanent hardness was treated with this compound and evaporated at pressure of 5 kgm., a copious fluffy precipitate resulted, while in a control experiment hard scale was produced on the walls of the vessel.—*R. E. Thompson.*

Bacteriological Examination of Water. A. Guillerd. Ref. Zentr. ges. Hyg., 17: 4, 1928; Wasser u. Abwasser, 25: 68. From Chem. Abst., 23: 4758, October 10, 1929. The French, Belgian, English, German, and American method are described. Special significance is attached to *B. coli* and fecal tests, to the differentiation of human and animal strains of coli and the number of coli colonies. Interpretation of results is also dealt with.—*R. E. Thompson.*

Supplementing the Gersbach Fecal Titer for the Certain Determination of Bacteria in Water. R. Kopp. Zentr. Bakt. 1927, II, 267–71; Wasser u. Abwasser, 25: 69. From Chem. Abst., 23: 4758, October 10, 1929. The Gersbach fecal titer test after 24 hours' use was considered sufficient for *B. coli* without confirmatory tests. After 48 hours the test is not as good. For exact work it is necessary to confirm coli with dextrose fermentation at 45°.—*R. E. Thompson.*

Qualitative and Quantitative Investigations on Industrial Wastes in Ivanovo-Vosnesensk. A. Levin. Gig. i. Epidem. (Russia), 7: 18–24, 1928; Wasser u. Abwasser, 26: 24. From Chem. Abst., 23: 4988, October 20, 1929. Wastes from cotton bleaching, dyeing, spinning, and weaving plants containing only twice their volume of river water are considered. Precipitation with lime gave best results as judged by the oxidizability of the effluent.—*R. E. Thompson.*

Bacteriological Investigation into the State of Pollution of the Clyde at Port Glasgow, Greenock, and Gourock. D. Ellis. J. Roy. Tech. Coll. (Glasgow), 2: 149–42, 1929. From Chem. Abst., 23: 4760, October 10, 1929. Study of the pollution of the River Clyde showed, in general, that the swift running tide in the channel effectively scoured and purified the deeper waters. The quiet, shallow, shore waters were but little affected by the tide and contained numerous saprophytic organisms such as *Cladothrix*, sulfur bacteria, *Beggiatoa* and Eumycetous fungi. Black mud containing hydrogen sulfide underlaid the shore waters while white sand was found in the deeper waters. No corrective measures are considered.—*R. E. Thompson.*

Two Laws for the Purification of Potable Water. N. Malishewsky. Gesundh. Ing., 52: 569–71, 1929. From Chem. Abst., 23: 4984, October 20, 1929. Streeter has shown that a mathematical relation exists between the bacterial count in raw water R and that in filtered water E, in the form of the equation $E = CR^n$, where C and n are constants characteristic of a given water supply. Also, it has been shown that double application of a given water treatment gives better results than the corresponding treatment given in one application.—*R. E. Thompson.*

Purification of Water Supply in a Country Town. A. Gordon Gutteridge and J. H. Varcoe. Commonwealth Eng., 16: 432–5, 1929. From Chem. Abst., 23: 4985, October 20, 1929. A report on the establishment of water purification for Shepparton, Victoria. The filter plant recommended is of the rapid sand type, comprising a mixing and aëration channel, sedimentation basin of 78,000-gallon capacity, 4 filter beds of a total area of 450 square feet and clear water basin of 12,750-gallon capacity. Provision is made for extension of the plant to an additional 50 per cent capacity, for addition of a coagulating chemical and for inclusion of chlorination apparatus.—*R. E. Thompson.*

The Disposal of Oil-Field Brines (A Preliminary Study). Ludwig Schmidt and John M. Devine. Bur. Mines, Reports of Investigations No. 2945, 17 pp. 1929. From Chem. Abst., 23: 4760, October 10, 1929. The successful use of ponds for evaporation in the district studied is limited to properties producing very small quantities of brine; ponds, however, are of great value for temporary storage. Recovery of common salt is uneconomical except in special cases. The diversion of oil-field brines into selected streams offers possibilities. A complete analysis of all brines that are allowed to enter streams should be made to insure absence of poisonous salts. The delivery of brine from a field to a selected stream can be accomplished by pipe lines, or a small tributary stream, if available, can be set aside for that purpose, providing fresh water can otherwise be supplied to those obtaining their water for human or stock consumption from the tributary stream. The disposal of the brines by returning them to a subsurface formation appears to be feasible in isolated instances, but great care must be used in attempting this method. Not only is there danger that the brine will migrate to fresh-water sands and pollute a potable water supply, but also there is an ever-present possibility that this brine may endanger present or future oil production. Also in Petroleum Times, 22: 207–8, 1929.—*R. E. Thompson.*

Influence of Radioactivity of Water on the Biological and Biochemical Action of the Cells of Lower and Higher Organisms. J. Stoklasa. Strahlentherapie, 29: 324–32, 1928; Wasser u. Abwasser, 26: 31–2. From Chem. Abst., 23: 4986, October 20, 1929. A great increase in the richness of the flora and fauna occurs in moderately acid radioactive waters containing at least 10 p.p.m. of oxygen. An accelerated development of green algae and nitrogen-assimilating bacteria occurs not only in water, but in radioactive soils and rocks as well. The author recommends baths in radio-active water containing sufficient amounts of oxygen for accelerating enzyme activity in humans. Living cells have a selective adsorption capacity for rays of the several radioactive elements.—*R. E. Thompson.*

Sampling Boiler Water for Testing and Guidance of Treatment. S. C. Page. Universal Eng., 50: 1, 25–7, 1929. From Chem. Abst., 23: 4986, October 20, 1929. A simple layout makes it possible to make competent test of boiler water.—*R. E. Thompson.*

The Bacteriological and Chemical Standards Employed in Water Analysis W. James Wilson. J. State Med., 37: 439–43, 1929. From Chem. Abst., 23: 4986, October 20, 1929.—*R. E. Thompson.*

The Waste from Paper Mills in Pommern. Möller. Fischereizeitung Neudamm, 31: 623, 1928; Wasser u. Abwasser, 26: 31. From Chem. Abst., 23: 4988, October 20, 1929. Paper mill, brewery, and sugar refinery wastes are thought responsible for much of the damage to fish in Pommern.—*R. E. Thompson.*

Synopsis of Waste Purification from Coke Plants and Related Industries. A. Reich. Der Stadt. Tiefbau, 1929, 59–63; Wasser u. Abwasser, 26: 25–6. From Chem. Abst., 23: 4988, October 20, 1929. The ore wash water from blast furnaces contains fine dust particles and dirt which are precipitated in a settling tank, the water being used again. The cooling water contains gas and carried-over flue dust, and is purified for re-use in 2 or more shallow aërating basins. The blast furnace waste waters contain, in addition to much dust, such poisonous substances as cyanide. The water is conducted to a pit or tank in which hang full sacks of ferrous sulfate. Neutralization is effected by sodium hydroxide addition followed by slight acidification with sulfuric acid. Sometimes the wastes are purified by the addition of milk of lime by the use of a patented process (Steuer system). Coke plant waste water contains soluble gases and coke particles. The latter are precipitated in settling tanks. Phenols may be extracted with benzol and the phenol recovered by neutralization with sodium hydroxide or by distillation. Another method of reclaiming phenol is to treat the water with an excess of ammonia and heat to 98° to form ammonium phenolate, which is decomposed by sodium hydroxide. A third method of purification is the biological one, whereby phenol in dilute concentration serves as a suitable medium for bacterial growth. An Emscher filter or aëration chamber treated with activated sludge is recommended for carrying on the treatment.—*R. E. Thompson.*

The Treatment of Waste from Beet-Sugar Plants. M. Gevemeyer. Gesundh. Ing., 52: 443–6, 1929. From Chem. Abst., 23: 4988, October 20, 1929. The waste from such a plant is very high in organic matter, much of which can be removed by means of mechanical clarifiers and separators. This applies especially to the washing water, etc. The cooling water is used for beet washing. Chlorination tends to prevent the exhaustion of dissolved oxygen in streams into which the waste flows.—*R. E. Thompson.*

Copper-Lined Steel Boiler. Kurt Kapler. Apparatebau, 41: 152, 1929. From Chem. Abst., 23: 5146, November 10, 1929. The copper lining prevents corrosion and the steel takes care of the pressure requirements.—*R. E. Thompson.*

Corrosion of Metals Under Cyclic Stress. D. J. McAdam, Jr. Proc. Am. Soc. Testing Materials 1929 (preprint), No. 40, 54 pp.; cf. C. A., 23: 2410. From Chem. Abst., 23: 4919, October 20, 1929. A discussion of the effect of stress, time, and number of cycles on corrosion in Severn River water and fresh water. The metals investigated included steels, Monel metal, and stainless iron. The net effect of cyclic stress on corrosion is shown graphically by "net damage" diagrams. The net damage diagrams indicate that for the metals investigated the stress-time cycle relationship is exponential in character. The relationship for ordinary steels in both waters can be represented by practically the same diagram. Stress cycles, however small, increase the corrosion rate of ordinary steels. There is apparently no limiting stress below which the effect of cyclic stress decreases suddenly. It is not known whether there is such a limiting stress for Monel metal or stainless iron.—*R. E. Thompson.*

Some Methods of Preventing Pipe Corrosion. Fr. Besig. Korrosion u. Metallschutz, 5: 99–110, 1929. From Chem. Abst., 23: 4917, October 20, 1929. A review of the modern methods for the prevention of corrosion of underground pipes, with special reference to corrosion by electrolysis.—*R. E. Thompson.*

The Decomposition of Alkaline Carbonates in Aqueous Solution. B. L. Vanzetti. Gazz. chim. ital., 59: 219–24, 1929. From Chem. Abst., 23: 4157, September 10, 1929. A preliminary note. A critical review of the literature, many references to which are cited, shows the unsatisfactory state of knowledge regarding the decomposition of alkaline carbonates in water. Experiments by the author indicate that on boiling, about 70 per cent of the sodium carbonate in 0.1 N and stronger solutions is decomposed in 120 hours, a definite law of decomposition being established.—[See following abstract]. *R. E. Thompson.*

The Decomposition of Alkaline Carbonate in Boiling Aqueous Solution. II. B. L. Vanzetti and A. Oliverio. Gazz. chim. ital., 59: 288–300, 1929: cf. previous abstract. From Chem. Abst., 23: 4903, October 20, 1929. An extension and elaboration of the previous article. A discussion of the nature of

the equilibria which are established in aqueous solutions of alkaline carbonates between the liquid phase and the supernatant gaseous phase leads to deductions which indicate the possibility of decomposition to a marked extent of the carbonates as a result of the application of heat (boiling solutions), when the gaseous phase is maintained free of carbon dioxide. Experiments showed that at various concentrations of sodium carbonate, viz., 0.2 to 0.4 N, up to 74 percent was decomposed into sodium hydroxide in about 6 days, and with 0.2 N potassium carbonate up to 65 percent was decomposed in from 5 to 7 days. The decomposition was inappreciable when carried out in vacuo, because of the relatively low temperature at which the solutions boiled and the consequently smaller hydrolytic decomposition. Under the conditions studied in the experiments the decomposition followed a very simple law, viz., that the proportion of carbonate decomposed is proportional to the square root of the time.—*R. E. Thompson.*

The Contamination of a Well Which Feeds into the Savona Water System Lines, with Ammonia and Nitrites from an Industrial Establishment. CANALIS, et al. Jg. moderna, 21: 161–8, 1928; Wasser u. Abwasser, 25: 276. From Chem. Abst., 23: 4987, October 20, 1929. The well, which had delivered water of excellent quality for 27 years, suddenly became contaminated with tar, ammonia, and nitrites from the effluent of a nearby newly-built manufacturing plant. Recommendation: A protective zone 100 meters in diameter around wells should be kept free from polluting agencies.—*R. E. Thompson.* .

The Chemistry of Water and the Distribution of the Plankton Organisms. WALTER RAMMNER. Naturw. Umschau; Chem.-Ztg., 18: 54–7, 1929. From Chem. Abst., 23: 5201, November 10, 1929. A general discussion, more biological than chemical.—*R. E. Thompson.*

Water Chlorination in the Fight Against Yellow Fever. P. BUNAU-VARILLA. Comptes Rendus, Acad. des Sci., 1928, 187: 1005; Tropical Diseases Bulletin, 1929, 26: 308; Public Health Eng. Abst. U. S., July 27, 1929, E856c. It is suggested that chlorination, by killing vegetable growths, will render water unfavorable for breeding of mosquitoes and thus help in suppression of yellow fever.—*M. H. Coblentz (Courtesy of the Department of Scientific and Industrial Research, Water Pollution Research Board).*

The Theory of the Chlorination of Water. A. WOJTKIEWICZ, E. MIOCHUSTIN, and E. RUNOW, Centralbl. Bakt. Parasitenk, 1929, 77: 21; Chim. et Indust., 1929, 21: 1176. Main points are, briefly, as follows. (1) Only about 2 percent of chlorine requirement is used up in destroying bacteria; rest is used to oxidise organic and mineral matter. It is impossible to determine directly the exact amount of chlorine fixed by bacteria. (2) A certain relation exists between quantity of chlorine absorbed by bacteria and their resisting its antiseptic action. (3) Presence of electrolytes has great influence on quantity of chlorine consumed, depending not only on their concentration, but also on chlorine concentration. In relatively concentrated solutions, order in increasing activity of principal ions is K, Mg, Ca, and NH_4; while, in

dilute solutions, inverse order prevails. (4) Salts of heavy metals in quantities insufficient to exert of themselves any toxicity increase the antiseptic power of chlorine, while, at still greater dilution, opposite effect is observed. Corrosive sublimate at concentration of 2×10^{-6} increases antiseptic power of chlorine; but decreases it at concentration of 2×10^{-9}. Silver, gold, platinum, etc. show the same phenomenon. (5) After-effect of chlorine on water is favorable for development of bacteria. (6) Chloride of lime has greater antiseptic power than chlorine at 38°C., but less, at 18°C.—*M. H. Coblentz* (*Courtesy of the Department of Scientific and Industrial Research, Water Pollution Research Board*).

Examination of New Chlorine Preparations for the Disinfection of Drinking Water. K. Dudulowa, Z. f. Desinfektion, 1929, 21: 124; Wass. u. Abwass., 1929, 26: 3, 84. Four new chlorine preparations were tested: (1) Hydrosept tablets prepared by Heyden, Radebeul, Dresden; (2) effervescent (Brause) tablets I and II; (3) Preparation A; (4) Preparation B. Preparation B in doses of 1.5 tablets to 0.5 litre of water gave good removal of bacteria in 15 minutes from time of complete solution. Further tests of effect of these preparations on pathogenic bacteria are necessary.—*M. H. Coblentz* (*Courtesy of the Department of Scientific and Industrial Research, Water Pollution Research Board*).

The Action of Carbonic Acid in Natural Water. Tillmans. Chem. Ztg., 1929, 53: 49, 482. Paper read at meeting of Verein Deutscher Nährungsmittelchemiker. Describes simple formula for calculating the combined carbonic acid in water, which, by altering the constant, may be used for any temperature. Action of water, with and without oxygen, on iron has been studied. Rate of solution of iron in oxygen-free waters varies with hydrogen-ion concentration; but in oxygen-containing waters, there is no such dependence. In presence of oxygen, carbon-dioxide-free water forms on iron protective coating, consisting of mixture of rust and calcium carbonate. Iron hydroxide readily absorbs free carbon dioxide. When water containing bicarbonate, but no free CO_2, has stood for some time in contact with ferric hydroxide, much carbonic acid is a dsorbed and water becomes supersaturated with calcium carbonate, which is then precipitated.—*M. H. Coblentz* (*Courtesy of the Department of Scientific and Industrial Research, Water Pollution Research Board*).

Water Resources Activities being Prosecuted in North Carolina by Water Resources and Engineering Division, North Carolina Department of Conservation and Development. C. E. Ray. Reprint from J. North Carolina Water and Sewage Works Association, 1928, 6. Deals first with general work of the Division as described in Biennial Report and then, in more detail, with stream-gauging activities, describing importance of work, methods, and results.—*M. H. Coblentz* (*Courtesy of the Department of Scientific and Industrial Research, Water Pollution Research Board*).

The Biennial Report of the Water Resources and Engineering Division to June 30, 1928. North Carolina Department of Conservation and Development. Report presents activities of the Division for biennium ending June 30, 1928, and outlines program for next biennium. It deals with stream-gauging, power studies, hydrological studies, coast, port, and water-ways investigations, stream sanitation and conservation, chemical water analysis, drainage, underground water investigation, mapping, and airports. Lists are given of gauging stations maintained in North Carolina and tables of measurements. Reports on water powers of several rivers have been prepared. Reference is made to formation, by arrangement between the Department and State Board of Health, of Stream Sanitation and Conservation Committee. Expanded programs of water analysis and drainage investigation are suggested. Complete list of publications of Department connected with water resources is given.—*M. H. Coblentz (Courtesy of the Department of Scientific and Industrial Research, Water Pollution Research Board).*

Organization of Metropolitan Districts. L. Pearse. Proc. Lake Michigan Sanitation Congress, 1928, 4: 3, 36. Account of development of co-operation between municipalities in matters of water supply and pollution. Different types of organization, authority given to each, and methods of financing are discussed. Several organizations are described and their powers and duties discussed. Includes bibliography and appendices giving 1917 Sanitary District Act in Illinois and amendments.—*M. H. Coblentz (Courtesy of the Department of Scientific and Industrial Research, Water Pollution Research Board).*

Liquid Flow Indicators. W. Smith, A. M. Warlow and S. R. Blades. E. P. 313, 105, Ill. Off. J. Patients, 1929, No. 2115, 3886. Arrangement is described for indicating, by means of alarm, or cut-out, failure of normal flow of water in a supply system. Water flows into container, which is attached to lever with counterpoise at other end balancing the container when full. When flow falls below normal, container empties, weighted arm, descending, depresses a plunger which closes electric circuit to operate alarm, or cut-out device.—*M. H. Coblentz (Courtesy of the Department of Scientific and Industrial Research, Water Pollution Research Board).*

Permutit Water Treatment with Special Reference to Circulating Regeneration. K. Hofer. Das Essener Heft: Kl. Mitt. der Landesanstalt für Wasser-, Boden-, u. Lufthyg., Supplement 5, 1927, 250. Reversible base exchange reactions are described, by which (1) water is softened by passing over permutit and (2) latter, when exhausted, is regenerated with salt solution. Permutit Co. have produced an easily regenerated material, Neo-Permutit, whose effect is not diminished by impurities in water. Hufschmidt system by which material is regenerated *in situ* by circulating salt solution, with saving in salt, in time, and in loss of permutit, is described in detail. Article is illustrated by diagrams. *M. H. Coblentz (Courtesy of the Department of Scientific and Industrial Research, Water Pollution Research Board).*

Water Softening Plant. Surveyor, 1929, 76: 146. At Marion, Ohio, chemicals are stored in two tanks, each 16 feet in diameter and 30 feet high, of 215 tons capacity, and with hopper bottoms to discharge into Browning feeders and slakers. Lime and soda-ash solutions are ejected through dip pipes into mixing tanks designed for retention period of two hours and fitted with motor-driven paddles. Better floc is obtained if soda-ash addition occurs only above level at which lime reaction has become complete, thus avoiding complex reactions which would occur if lime and soda-ash were added at same time. Coagulated water passes through tank to Dorr clarifier, which reduces turbidity to 2 p.p.m., thence through another tank to settling tank, and subsequently to carbonating tank and final soft water storage basin. Carbonating in carbonation tank having resulted in fine particles passing to soft water tank, treatment was carried out in feed well of settling tank in order to allow time for fine calcium carbonate to settle. Maximum reduction of alkalinity was finally obtained by carbonating in both tanks. Alkalinity is reduced from about 55 to 35 in first, and further to about 30 p.p.m. in second tank.—*M. H. Coblentz (Courtesy of the Department of Scientific and Industrial Research, Water Pollution Research Board).*

A New Apparatus for Water Sterilization by Ultra-Violet Light. L. WAMOSCHER. Cbl. für Bakt., Parasitenk., and Infectionskrankh. Ref. 1928, 90: 15-16, 377. Brief history of earlier work with mercury vapour lamps ushers in description of new apparatus in which water is exposed to rays in thin layers. Lamp construction is described in detail. In experiments with Berlin tap water to which 500,000 bacteria per cubic centimeter had been added, *B. coli* proved most resistant; but with voltage of from 100 to 140 and current of from 3.6 to 4.5 amperes, water could be completely purified at rate of 6 litres a minute. Taste was unaltered; temperature made little difference; no trace of hydrogen peroxide was detectable. Filtered Spree river water to which typhoid and coli bacilli had been added even at flow of 4 litres per minute was not completely sterile, certain saprophytic bacteria being very resistant; but in no case did pathogenic bacteria survive. More information as to wavelengths actually effective is required. In discussion, effects of salts, organic matter, suspended matter, and physical condition were mentioned and methods of control compared with those in chlorination.—*M. H. Coblentz (Courtesy of the Department of Scientific and Industrial Research, Water Pollution Research Board).*

Preliminary Report on the Chemical Quality of the Surface Waters of North Carolina with Relation to Industrial Use. C. E. RAY and E. E. RANDOLPH. N. Carolina Department of Conservation and Development. Economic Paper. No. 61. Introduction briefly reviews economic position in regard to electric power, transport, labor, and education. In Part II, rainfall is considered. Mean annual rainfall is between 45 and 55 inches in Piedmont district and in Coastal Plain and about 80 inches on easterly slopes of Appalachians. Gauging stations have been established on majority of rivers, and records of stream flow are published by the Water Resources Division of the Department. In Part III, the chemical analyses of surface waters are tabulated under three

groups, (1) Mountain; (2) Piedmont Plateau; and (3) Coastal Plain. Mountain waters are very clear, with average turbidity of 30 p.p.m., color of 14 p.p.m. and total dissolved solids of 30 p.p.m. Many industries requiring abundant supplies of excellent water are established here, e.g., paper mills, bleaching and textile works. Piedmont Plateau, however, is regarded as chief industrial area; waters are of good quality, with average hardness of 20 and turbidity of 102. Rivers of Coastal Plain fall into two groups, those that flow from Plateau, and those that rise in deposits of Coastal Plain. Only few analyses have been made of these waters, but enough to show that they are pure, with average hardness between 15 and 30 and dissolved solids between 55 and 70 p.p.m. Ground waters of this section are also of great importance and have been extensively studied by Geological and Economic Survey. Superiority for industrial purposes of N. Carolina waters over those of other states is due to their freedom from hardness and suspended matter. In Part IV, relation between chemical composition of water and its industrial use is discussed. Three maps of N. Carolina are appended, showing stream gauging stations, water sampling stations, and power stations.—*M. H. Coblentz* (*Courtesy of the Department of Scientific and Industrial Research, Water Pollution Research Board*).

The Ruhr Reservoirs Association (Ruhrtalsperrenverein): the Möhne and Sorpe Valley Reservoirs. E. LINK. Das Essener Heft. Kl. Mitt. der Landesanstalt für Wasser-, Boden-, u. Lufthyg., Supplement 5: **1927**; 185. Ruhr serves as water supply for its own and other districts. Various methods adopted for collection are described. Initiated in 1898 by voluntary association for conservation of supplies, legal status was acquired in 1913. Its program is reservoir construction in valleys of tributaries. Description is given of work of Association, including detailed accounts of great Möhne valley reservoir and of proposed Sorpe valley reservoir.—*M. H. Coblentz* (*Courtesy of the Department of Scientific and Industrial Research, Water Pollution Research Board*).

The Water Supply of the Ruhr Coal Region. H. BRUNS. Das Essener Heft. Kl. Mitt. der Landesanstalt für Wasser-, Boden-, u. Lufthyg., Supplement 5, **1927**: 197. General discussion, dealing with quantity required, technical problems, and, principally, hygienic control. Supply comes from Ruhr and Lippe, from pit water, and from catchment reservoirs and amounts to from 700 to 750 million cubic metres per annum. Separate supply for heavy industrial consumption is neither hygienically, nor economically, advisable. Water as supplied has proved uniformly satisfactory, even, in some cases, without further softening, for boiler feed. Greatest difficulty has been aggressive carbon dioxide. Ruhrtalsperrenverein catchment reservoirs and other methods of obtaining supplies are discussed, with special reference to "ex-filtration" from rivers and to bacteriological control. Chlorination is controlled by daily bacteriological examination. Great diminution in typhoid is ample evidence of adequate sanitary control both of water supply and of sewage disposal.—*M. H. Coblentz* (*Courtesy of the Department of Scientific and Industrial Research, Water Pollution Research Board*).

The Water Supply of the City of Essen. B. NERRETER. Das Essener Heft. Kl. Mitt. der Landesanstalt für Wasser-, Boden-, u. Lufthyg., Supplement 5, 1927: 219. History of water supply of Essen and of investigations made for purpose of supplementing it, including account of water supplies and of collection methods in Ruhr district generally. Author discusses comparative values of filtered water, of ground water, and of "ex-filtered" river water and describes chlorination methods practised. Detailed description is given of plant, built between 1912 and 1915. Passing from river through preliminary clarification and settling basins into "ex-filtration" basins, water percolates down to gravel, where it is collected by seepage galleries, or wells, and pumped to storage well and thence to high reservoirs, commanding filtration plant. Composition of water, pumping machinery, and tests of works are fully treated. Article is illustrated.—*M. H. Coblentz* (*Courtesy of the Department of Scientific and Industrial Research, Water Pollution Research Board*).

Torquay Corporation Waterworks. R. V. TOMS. Surveyor, 1928, 73: 697. Paper read before Institution of Water Engineers. In 1913, *Synura* trouble was successfully overcome with copper sulphate. One pound per million gallons was applied from bags drawn behind motor boat, thorough mixing ensuing from propeller action. In 1910, mains became infested with "pipe moss" (polyzoon, *Plumatella emarginata mucosa*). Pabulum was cut off by installation of 12 Candy mechanical filters with satisfactory results. Filter plant is briefiy described. All service reservoirs have been covered in; all but one, with reinforced concrete. Latter, to avoid excessive floor pressure, was roofed with steel girders in single span.—*M. H. Coblentz* (*Courtesy of the Department of Scientific and Industrial Research, Water Pollution Research Board*).

Alor Star Emergency Water Supply. C. H. DOBBIE. Water and Water Engineering, 32: 379, 305–308, July 21, 1930. Water for Alor Star, capital of Malay State of Kedah, is obtained from two streams having catchment area of three square miles in low hills about 24 miles distant. Only storage available is one reservoir of 1,000,000 gallons capacity. Inadequacy of supply for 25,000 people had been for some time apparent and plans made for increasing it; but before they could be completed, remarkable drought set in which necessitated immediate action. Daily flow, which never before had been less than 170,000 gallons fell in February 1930 to 30,000 gallons. Cholera has ravaged this district in the past and immediate additional supply of good water was imperative in order to avert another cholera outbreak. Existing supply main flows along the Anak Bukit River, which has a considerable bow, but is very polluted. On January 28 steps were taken to obtain a supplementary supply of purified water from this river and by February 9 a pumping plant with tanks for alum treatment and sterilization had been installed. On February 11 the treated water was tested and approved by health authorities and pumping into service main commenced. From first day, system proved pronouncedly successful.—*Arthur P. Miller*.

The Selection and Operation of Pumping Machinery for Water Works. F. E. F. DURHAM. Water and Water Engineering, 32: 379, 308–317, July 21, 1930. In this section specifications are taken up. The perfect specification has not yet been written. Author believes that in addition to stating duties which machinery has to perform and such details of design and construction as writer's knowledge permits, a useful purpose would be served by describing also what may be called the indefinite requirements. Misunderstandings may be avoided by supplementing specifications with general arrangement drawings, indicating layout of pumping plant and any special details required. Engine failures and costs and efficiencies of various coals are also discussed. Concludes with brief discussion of purchasing of stores for large organizations and of keeping of records on operation of machinery.—*Arthur P. Miller.*

New Water Tower at Hertford. S. M. SENIOR. Water and Water Engineering, 32: 379, 317–319, July 21, 1930. Tower is in center of small wood close to rapidly growing residential area in Bengeo. Appearance, therefore, was of primary importance. Tank is supported by eight columns of generous proportions and octagonal central shaft. Elimination of horizontal bracings, arches between the columns, and treatment of outer surfaces add greatly to appearance of structure. Floor of tank is 75 feet above ground level and structure is designed to withstand wind pressure of 30 pounds per square inch. Tank capacity is 200,000 gallons; it is divided concentrically into two equal parts. Cupola with domed roof and flagstaff encloses small portion of tank roof over central shaft. Access manholes are fixed in roof, one in cupola to inner tank and one on roof of outer tank.—*Arthur P. Miller.*

The Effect of Surface Waves on the Discharge over Weirs. Prof. ARNOLD HARTLEY GIBSON. Water and Water Engineering, 32: 379, 319, July 21, 1930. Experiments were made on weirs of four types, namely, suppressed rectangular, rectangular with two end contractions, 90-degree vee notch, and broad-crested. Heads ranged up to 6½ inches over rectangular weirs and to 17 inches over vee notch; wave-heights ranged up to 8 inches. Wave disturbance increases discharge over a weir, magnitude of effect depending upon various factors. With waves height of which does not exceed mean head on weir-crest and with normal type of measuring-weir, a correction may be applied. Effect of waves on discharge is less over broad-crested than over thin-crested weir and is more irregular.—*Arthur P. Miller.*

Water Supply from Boreholes. H. T. BURGERS. Water and Water Engineering, 32: 379, 320–324, July 21, 1930. About fifty years ago first practical well-boring apparatus was imported into South Africa. This portable field apparatus utilized method known as percussion drilling. String of tools weighing 1,800 pounds or more is lifted by machine and allowed to drop anywhere from 24 to 46 inches. The bit on bottom end of string does the cutting. In soft material, borehole will be sunk from 40 to 60 feet per day, but in very hard rock one to two feet a day is considered satisfactory. Rotary shot drills have become popular lately. Action in this system is abrasive. Steel shot is poured into borehole, while steel head of annular section revolves with

controlled pressure on the shot. Verticality and uniform diameter of borehole are important. Unless borehole is straight, rods of deep well pump will drag on sides with loss of durability and efficiency. To exclude extraneous materials from borehole, it must be cased or lined to solid rock. Strainers are designed to prevent entry of sand and admit water. Black wrought iron pipe properly coated and inserted new has unknown length of life in a borehole. Little thought need be given to possibility of casing being crushed by external pressure in boreholes under 500 feet deep. Yield is a matter of much speculation. Present yield can be ascertained by pumping to exhaustion. Formulae for ascertaining yield are discussed.—*Arthur P. Miller*.

Bartley Reservoir—Birmingham (England) Corporation's New Storage Facilities. Anon. Water and Water Engineering, 32: 380, 363–365, August 20, 1930. Envisaging possible interruption by breakage of supply by Elan aqueduct from Frankley reservoir, new 500,000,000-gallon Bartley reservoir adjacent to city was designed. It also serves in equalizing capacity over peak periods. It is of impounding type, made by building embankment across a valley. Embankment is earthwork, with central core-wall of reinforced concrete. Accessory pipes and valves connecting reservoir with Frankley filters are briefly discussed. Reservoir site was cleared of trees, hedges, and other soil and vegetable matter, and entire basin then covered with stone ballast or concrete to preserve purity of the water and to prevent erosion of underlying marl by wave action. Intercepting channels divert from reservoir all surface water.—*Arthur P. Miller*.

The Arkley (England) Reservoir. Anon. Water and Water Engineering, 32: 380, 375–376, August 20, 1930. Is of mass concrete construction, with capacity of 5,000,000 gallons. Roof was covered with three inches of gravel before earth was placed on it and six-inch drain was then constructed along eastern wall. Gravel stratum permits drainage of entire roof down to the single collector. Several cracks developed in walls, due to weather conditions, and slight settlement occurred. Cracks were grouted with cement under pressure and asphalt fillet, retained by 4½-inch brick wall, carried round in angle of reservoir.—*Arthur P. Miller*.

The Steam Turbine as Applied to Borehole Pumping. J. F. Haseldine. Water and Water Engineering, 32: 380, 366–373, August 20, 1930. Selection of type of prime mover depends to certain extent on local conditions. In case of Roestock pumping station of Barnet (England) District Gas and Water Company, steam, being already available, was adopted. Borehole had maximum diameter of 36 inches and duty of 1,500,000 gallons per day, from depth of 200 feet, to be discharged into mains against total maximum head of 550 feet, was required. Vertical spindle steam turbine driving vertical spindle turbine borehole pump was adopted. Details of design of pump and turbine are discussed, as well as their installation.—*Arthur P. Miller*.

Service Reservoirs for Lossiemouth (Scotland) Waterworks. Alexander Grant. Water and Water Engineering, 32: 380, 377–378, August 20, 1930.

Consist of two circular tanks, one over the other. Lower tank, 74 feet in diameter, holds 500,000 gallons and upper, 40 feet in diameter, 100,000 gallons. —*Arthur P. Miller.*

Water Supply for Fire Purposes. G. Baxter. Water and Water Engineering, 32: 380, 380–382, August 20, 1930. Statutory (British) obligations of water authority are set forth, as well as necessary sizes of mains and types and sizes of hydrants and tanks; pumping from docks and streams is discussed.—*Arthur P. Miller.*

Asbestos Cement Pipes—"Italit" Piping. Anon. Water and Water Engineering, 32: 379, 324–325, July 21, 1930. "Italit" pipe made from asbestos fibre and hydraulic cement weighs only from one-third to one-fifth as much as cast iron pipe having same standard specifications. It has many advantages in connection with transportation of water, oil, and gas under pressure.—*Arthur P. Miller.*

Copper Sulphate Clears Algae Trouble at Winslow. D. E. Lewis. Municipal Sanitation, 1: 8, 442, August, 1930. Storms followed by warm sultry weather induced exuberant growth of algae in the impounded water supply which, in turn, infected 5,000,000-gallon concrete reservoir supplying the town, creating unpleasant tastes and odors strenuously objected to by consumers. Reservoir was drained, thoroughly cleaned, and side walls well washed with copper sulphate; after which it was re-filled and dosage of 8 pounds copper sulphate per m.g. applied at influent. This treatment eliminated the algae trouble. As safeguard against possible presence of pollution, chlorination was instituted and adjusted to show from 0.2 to 0.4 p.p.m. residual chlorine at taps. Algae present were mainly *Protococcus*. Cost of cleaning reservoir was $230.—*R. E. Noble.*

The Sanitation of Summer Camps. Joel I. Connolly. Municipal Sanitation, 1: 8, 445, August, 1930. Comprehensive article including section entitled **A Safe Water Supply Essential.** Importance of supply of pure water for drinking and culinary purposes is pointed out. It is best to secure water which requires no treatment if it be available in sufficient quantities. Deep wells are often suitable. Great care must be taken to prevent contamination by leakage around top of well. Location and construction must of course be such as to preclude any possibility of contamination by camp sewage. About a month or three weeks before camp opens, water supply should be sampled, using great precautions to assure representative sample, sterile containers, and proper technique of collection. The sampler should be carefully instructed by an experienced person, in order to avoid the misgivings and worry attendant upon a bad report which might be caused only by improper collection of sample. The bacteriological examination, made in accordance with Standard Methods of American Public Health Association and American Water Works Association should show compliance with standard set by United States Public Health Service for water supplies used by common carriers engaged in interstate traffic.—*R. E. Noble.*

Some Higher Court Decisions of Interest to Sanitarians. Draining Water to Another's Land. LEO T. PARKER. Municipal Sanitation, 1: 8, 457, August, 1930. It has been held by numerous higher Courts that owner of property cannot recover damages resulting from water drained from higher land. Most recent case involving this point of law is SCHWARTZ v. Wapello County, 227 N. W. 91. Generally speaking, owner of real estate is entitled to recover compensation from a city, or county, where testimony clearly proves that he has been damaged by water overflowing his land as result of defective sewer construction; or where water drains over his property owing to negligence of municipal or county employees. Case cited is Floyd County *v*. FINCHER, 150 S. E. 577.—*R. E. Noble.*

The New Michigan Law with Regard to Stream Pollution and Sewage Disposal. WALTER A. SPERRY. Municipal Sanitation, 1: 3, 157, March, 1930. Act 337 of 1927 authorized Department of Conservation to preserve and to guard against pollution of lakes and streams and to enforce all laws relating thereto, encouraging protection and propagation of game and fish. Act 157 of 1927, amended as Act 126 of 1929, Section 4-F Part 4, is an amendment to the Home Rule Act empowering cities coming under that Act to own, operate, and levy rates for use of sewage disposal systems. A Waste Disposal Bill, granting somewhat similar but broader powers to all Michigan cities and villages, resulted in Act 320 of 1927, later amended as Act 160 of 1929 to include also garbage disposal and to make it possible for counties, or for groupings of two or more communities or townships, to form sanitary districts clothed with same powers as were granted cities and villages in original Bill, for operation of sewage and garbage disposal plants. The Metropolitan District Act 312 of 1929 empowers communities to combine to form metropolitan districts and provides the necessary legal machinery therefor. Act 245 of 1929 created Stream Control Commission, to correlate and administer all problems relating to public waters encountered by constituent bodies composing the commission. It has power to control the pollution of any waters of State, or of the Great Lakes areas within the State boundaries, to make rules and regulations governing same, and to prescribe the powers and duties of the commission in prohibiting, under appropriate penalties, the pollution of any waters of the State. Article includes interesting statistics relative to growth of urban communities in Michigan.—*R. E. Noble.*

Waterbury Acts to Prevent Pollution of Domestic Water Supply. Municipal Sanitation, 1: 6, 336, June, 1930. Newly adopted local regulation of Waterbury, Conn., reads: "No flush valve that is installed to flush water closets shall be connected directly to any city water pipe or service pipe in this city, unless such flush valve is provided with proper air breaks to overcome the vacuum created when the water is shut off, and all such valves shall first have the approval of this (Plumbing Inspection) Department."—*R. E. Noble.*

Pool Cross-Connection Outlawed. Municipal Sanitation, 1: 6, 336, June, 1930. California has outlawed swimming pool cross-connections. State Board of Public Health on April 12, 1930, adopted following regulation: "Wa-

ter added to any swimming pool, or swimming pool piping system, anywhere along its course, from a public water supply system, shall be added overhead with a free overfall and no cross-connection shall be made between the swimming pool piping and the public water supply piping. This regulation applies equally to public and private swimming pools."—*R. E. Noble.*

Keeping B. Coli Down at Ocean Bathing Beaches. W. T. KNOWLTON. Municipal Sanitation, 1: 3, 137, March, 1930. California State Department of Health some time ago adopted count of 10 *B. coli* per cubic centimeter as permissible limit of contamination, not only in determining areas of fields of pollution, but also wherever beaches are used for recreational purposes, such as swimming, fishing, boating, and camping, or for picnics.—*R. E. Noble.*

Can a Public Pool Be Safe? CARL A. HECHMER. Municipal Sanitation, 1: 3, 138, March, 1930. Author describes safe, modern, attractive swimming pool located at Chevy Chase, Md., privately owned and operated, free from cross-connection dangers, and with drinking-water quality maintained by recirculation, filtration, and chlorination. Residual chlorine is checked every 2 to 3 hours and *Bact. coli* tests are made 2 to 3 times weekly by engineer and once weekly by State Health Department. During 1929 operating season, 92 percent of samples examined met U. S. Treasury Department standard. Article is amplified by photograph, pool plan, and operating forms.—*R. E. Noble.*

Sanitary Control of Pools in New York City. JOHN E. DOWD. Municipal Sanitation, 1: 4, 216, April, 1930. Pre-season inspection is made of all equipment, particularly that for filtration and sterilization and for keeping pool clean. Approved methods of control are recommended, apparatus for making tests supplied, and instruction given in method of making tests. Samples are collected systematically for *Bact. coli* tests by A. P. H. A. Standard Methods and for occasional hemolytic bacteria and total count tests with blood agar plates. In 5 samples of 120, during past season, presence of *Bact. coli* was confirmed, notwithstanding residual chlorine content ranging from 0.2 to 0.4 p.p.m. Over-all efficiency limit was regarded as being from 0.25 to 0.3 p.p.m. Data are submitted to show that, apparently, it is possible to keep pools in good sanitary condition under severest conditions with careful operation and continuous maintenance of adequate excess chlorine. As standard, it is suggested that *B. coli* should never be present in 1.0 cc. and only occasionally in 10.0 cc. This is in line with standards of Joint Committee of A. P. H. A. promulgated in October, 1927.—*R. E. Noble.*

Policing Swimming Pools in the Laboratory. Municipal Sanitation, 1: 4, 217, April, 1930. Regulations of Sanitary Code of State of Connecticut applying to swimming pools require:— (1) that not more than 10 percent of samples covering a considerable period of time shall show bacterial counts exceeding 100 per cubic centimeter and not more than 2 out of 5 consecutive samples collected at different dates shall be positive for *B. coli* in 10 cc. (2) that whenever alum treatment is in use, pool water shall at all times show an

alkaline reaction. State Department of Health advises maintaining residual chlorine content of from 0.3 to 0.5 p.p.m. while pools in operation and, since acid water may cause sore eyes, pH of from 7.0 to 7.6.—*R. E. Noble.*

Problems in Cross-Connections. WM. C. GROENIGER. Municipal Sanitation, 1: 5, 250, May, 1930. Most difficult task in matter of cross-connections is perhaps teaching the skeptical and indifferent the how and why of siphons and siphonage. Definition and principle of siphons are given in detail and illustrated by diagrams of various common plumbing fixtures often concerned in cross-connections. Problem is to prevent the flowing back by siphonage from a plumbing fixture into the water distributing system of water, excrement, urine, or wastes. Two methods are available to correct faulty installations without need of replacing existing bowl and valve: (1) air-break in flush pipe; air-intake opening of which must be at least one inch above top of closet bowl; (2) automatic air-intake valve, or vacuum valve, on fixture branch. There are today many hospital sterilizers for instruments, utensils, water and bed pans, as also integral supply lavatories, water closets, and drinking fountains, which by gravity alone, without intervention of siphonage, may return sewage and wastes into distribution system.—*R. E. Noble.*

Loss of Residual Chlorine in a Swimming Pool. W. L. MALLMAN and O. B WINTER. Municipal Sanitation, 1: 7, 382, July, 1930. Great care should be exercised in cleaning around swimming pools to avoid introduction into pool of foreign material, particularly of cleaning powders containing carbonates. It is based on the following conclusions from an experiment. The authors found that: (1) scrub water slopped into swimming pool resulted in loss of residual chlorine content. (2) The cleaning powder used, added experimentally to chlorine-treated water, caused loss of residual chlorine as measured by the *o*-tolidine test and growth of *B. coli.* (3) Sodium carbonate was the only constituent of the cleaning powder that caused loss of residual chlorine.—*R. E. Noble.*

Supervision of Institutional Water and Sewage Plants. A. W. BLOHM. Municipal Sanitation, 1: 7, 395, July, 1930. A supervising engineer from the Bureau makes monthly visits to operating water filtration plants and sewage treatment plants designed by Bureau of Sanitary Engineering, Maryland State Department of Health. Cost of such supervision is borne collectively by the plants. Most plant operators have found as a result that, in addition to being enabled better to operate their plants, they have been able to obtain necessary improvements, materials, or manual assistance, more readily than formerly. Specific instances of such benefits are described.—*R. E. Noble.*

Safeguarding Swimming Pool Water. G. T. LUIPPOLD. Municipal Sanitation, 1: 8, 455, August, 1930. In several of the larger cities, the City Health Department exercises a systematic and regular inspection and collection of samples for bacterial analysis. Recirculation of pool water through filters with subsequent sterilization, usually by chlorine, constitutes an additional safeguard. Many states have adopted standards laid down in report of Joint

Committee on Bathing Places of American Public Health Association at Conference of State Sanitary Engineers. (1) Not more than 10 percent of samples covering any considerable period shall contain more than 100 bacteria per cubic centimeter, when plated on agar, or litmus lactose agar, for 24 hours at 37°C. (2) No single sample shall contain more than 200 bacteria per cubic centimeter.—*R. E. Noble.*

Water Supply Samples for Examination by the Laboratory. JACK J. HINMAN, JR. Water Works Eng., **84**: 1, 19, January 14, 1931. Gives detailed instructions on preparation of water samples for bacteriological analysis. Volume collected is so minute, compared with total supply, that it is difficult to obtain representative sample. Sterilization of container should be accomplished in hot-air oven, or autoclave. Instructions of laboratory should be followed, and sufficient data supplied for proper interpretation of the analysis. Samples should be shipped promptly after collection and at least 48 hours before Sunday, or a holiday.—*Lewis V. Carpenter.*

Some Notes on Performance of a Large Steel Water Main. MAJOR F. JOHNSTONE-TAYLOR. Water Works Eng., **84**: 1, 49, January 14, 1931. Coolgardie main in Western Australia extends for 350 miles and from 2 to 5 million gallons per day are pumped against total head of about 270 feet through the 30-inch steel pipe-line by 20 300-h.p. pumps of Worthington non-rotative high-duty class. During period 1905–1908, friction increased, due to tuberculation. Water was deaërated and 4 g.p.g. of lime added; but this caused troublesome lime deposits. Experiments showed that bituminous coating disintegrated on inside of pipe, cracking in irregular lines termed "alligatoring." These cracks later are starting points for corrosion. Leaks due to external corrosion have been stopped by driving in wooden plugs which are sawed off flush with plate, a steel band being put around pipe with pad of rubber insertion covering hole. External corrosion occurs mostly in soils impregnated with salt. General conclusions, after 20 years of operation, point to corrosion trouble being mainly due to presence of mill scale and to defective coating.—*Lewis V. Carpenter.*

Water Rights Do Not Always Permit Diversion of Streams. LEO T. PARKER. Water Works Eng., **83**: 26, 1839. December, 1930. Courts refused city of Pueblo right to divert natural stream for city supply because of damage to property owners downstream. Private company developed an underground supply, and town acquired a small tract of land near tracts of said company; court held that town had legal right to do so, as private company had scarcity of water before this new work was started. Court decision stated that water company having made contract to furnish water to riparian owner for $1000 was relieved of contract when said riparian owner transferred his property to another. When a natural stream flows through a property the owner of land on lower reaches of stream is entitled to natural flow of stream. Upper owner must use reasonable care not to damage property of lower owner and if he so does, he is liable for damages. Courts held that city had power to prevent drilling of oil and gas wells near wells for city supply, as this was one of the few supplies available.—*Lewis V. Carpenter.*

To What Degree is the Water Works Liable for Personal Injuries? LEO T. PARKER. Water Works Eng., 84: 2, 94, January 28, 1931. A city cannot pass valid laws which would relieve it, or a private water company, from liability for injuries to pedestrians caused by negligence of employees in construction and maintenance of water works systems. Testimony must be introduced proving that negligence of city officials, or employees, resulted in injury, in the case of adults. Cites case of woman stepping into a hole surrounding a water plug and being seriously injured; city was held liable because hole had been there long enough to have been repaired. Negligence is not necessary in the case of children. A city is liable in damages for injury sustained by child who is attracted by any appliance, machine, or the like, left standing in place convenient for a child to play. Various courts have also held that public officials are not responsible for negligence of those employed under them, if the officials have used ordinary care to employ persons of suitable skill. Review of recent higher court cases indicates that municipality is not liable for damages sustained by automobile drivers who drive into street excavations, if warnings, or barriers, are capable of being observed by reasonably careful and prudent drivers. It is the duty of a contractor to safeguard pedestrians: principal contractor is not liable in damages for injury caused by a sub-contractor. Author gives a good differentiation between an official and an employee. Court has held that contractor, or city, must furnish safe place to work, as well as safe tools.—*Lewis V. Carpenter.*

Thawing Services by a Self-Contained Gasoline-Electric Apparatus. E. G. WILSON. Water Works Eng., 83: 26, 1835, December, 1930. Apparatus, designed to thaw pipes from one-half inch to six inches in diameter, is self-contained, portable, and available for service in all parts of town, which was not the case with transformer method. The gasoline engine is direct-connected to specially built 10-kilowatt alternating type generator, delivering current of 400 amperes at 30 volts. Costs were reduced from $6.05 to $1.78 per service.—*Lewis V. Carpenter.*

Small Town Builds Its Own $70,000 Water Supply System. FREDERICK C. WILLIAMS. Water Works Eng., 84: 289, January 28, 1931. Detailed costs are given for construction of water supply system for Cumberland, R. I., consisting of pump house on banks of a pond, 10-inch suction line 360 feet long, and stand pipe 22 feet in diameter and 125 feet high. Water comes from impounding reservoir and distribution system is made up of 6-inch, 8-inch, and 10-inch pipe. Chlorination is the only treatment. Total cost of improvement was $64,646.25—*Lewis V. Carpenter.*

Technique of Steam Pollution Investigations. F. W. MOHLMAN, S. L. HERRICK, and H. GLADYS SWOPE. Ind. Eng. Chem., 23: 209–13, February, 1931. Special considerations are to be noted in the sampling of a polluted river. A carefully chosen sampling program is to be followed and the most representative points in the stream used for the testing stations. Bottom samples should be carefully collected to assure the elimination of sludge deposits. Composite samples are a necessity for correct data. Interesting

diagrams locating proper sampling stations are given.—*Edward S. Hopkins (Courtesy Chem. Abst.).*

Effect of Sunlight and Green Organisms on Reaëration of Streams. William Rudolfs and H. Heukelekian. Ind. Eng. Chem., 23, 75–8 January, 1931. Laboratory study, based upon the dissolved O, Bio-chemical O demand, and *B. coli* tests of samples collected from Delaware River for 24-hour periods. Results presented in chart form indicate that dissolved O increased rapidly during forenoon, reaching a maximum between noon and 4 p.m. During night, decrease was noted until lowest point was reached at 3 a.m., with subsequent increase again after daylight. This occurred in spite of temperature decrease during night hours. Many experiments confirm the belief that this O is given off by blue-green algae growths in vigorous activity. These data have important bearing on steam pollution surveys, particularly in summer, and discredit afternoon sampling if these algae are present.—*Edward S. Hopkins (Courtesy Chem. Abst).*

Some Inter-relationships of Plankton and Bacteria in Natural Purification of Polluted Water. C. T. Butterfield and W. C. Purdy. Ind. Eng. Chem., 23, 213–8, February, 1931. Normal polluted water contains many bacteria and plankton; if these removed, or killed, oxidation ceases. Studies with *B. aërogenes* and with bacteria-free *Colpidium* in pure culture were undertaken to show the relationship between them. Active multiplication of bacteria caused rapid depletion of dissolved oxygen until a limiting bacterial concentration was reached. Pure plankton cultures gave moderate oxygen depletion; admixture of the cultures gave not only rapid initial oxygen absorption during period of bacterial multiplication, but also continuous absorption, even after limiting concentration of colpidium had been reached. This indicates that function of plankton in polluted streams is to keep down bacterial population below their limiting value and thus provide suitable conditions for their multiplication, thereby giving more complete oxidation.—*Edward S. Hopkins (Courtesy Chem. Abst.).*

Precautions Needed in the Ammonia-Chlorine Treatment of Swimming Pools. A Preliminary Study. Lyle L. Jenne and Henry R. Welsford. Ind. Eng. Chem., 23: 32–4, January, 1931. Use of chloramine for sterilization makes possible high residual Cl content without unpleasant taste, odor, or irritating effects, of free Cl. With re-circulation of swimming pool water as generally practised, the NH_3 content is to some extent cumulative and oxidation to nitrite occurs in certain concentrations. These products interfere with color test for available Cl. Use of an anti-chlor will destroy chloramine. Any color formed after dechlorination should be deducted to obtain true Cl value. The apparently fallacious results thus obtained at times do not outweigh advantages of this treatment.—*Edward S. Hopkins (Courtesy Chem. Abst.).*

Boiler Water Conditioning: A Pittsburg Development. J. N. Welsh and H. A. Jackson. Proc. Eng. Soc. West. Penn., 46: 327–42, 1930. Success in boiler operation at high pressures depends upon timely and correct analysis

of the constantly changing boiler water. Fundamental requirements of boiler water conditioning are (1) that sufficient concentration of carbonate, or of phosphate, ions must be maintained to prevent precipitation of $CaSO_4$; (2) that all organic and suspended material must be eliminated; and (3) that minimum concentration of solids in feed water must be maintained. Numerous specific examples from Pittsburg area of such conditioning are cited.—*Edward S. Hopkins (Courtesy Chem. Abst.).*

Selecting the Materials to Prevent Clogging. FRANK WOODBURY JONES. Civil Eng., 1: 118–20, 1930. The size and durability of filter bed material must be such as to insure an abundant air supply through the entire mass. Surface growths seal the beds, diminish the air supply and cause "ponding." This is also caused by application of improperly treated sewage. Disintegration of the beds may cause clogging in the depths decreasing the voids. This may also be caused by the rough surfaces of the aggregates retaining the solids preventing unloading. Uniform particles of clean material should be used regardless of size. They must be as durable as possible to retard disintegration by weathering.—*Edward S. Hopkins (Courtesy Chem. Abst.).*

Use of Chloramine in Treatment of Pool Water. J. F. T. BERLINER. Beach & Pool Magazine, 5: 9–10, 1931. Chloramine, unlike chlorine, is not destroyed by organic matter, or by actinic rays, nor is it removed by aëration. Less chlorine is required; a reduction of as high as 80 per cent having been accomplished. Complaints of tastes, odor, and irritating effects, as well as algae and slime growths are reduced. Ammonia is added before the chlorine. Before applying *o*-tolidine test, nitrites, if present, must be destroyed by oxidation with H_2O_2.—*G. L. Kelso (Courtesy Chem. Abst.).*

A Routine Method for Direct Determination of *B. coli* in Large Quantities of Water on a Solid Medium. F. DIÉNERT and P. ÉTRILLARD. Annales des Services Techniques d'Hygiène de la Ville de Paris, 1930. In France, customary test for *B. coli* is with 0.1 percent phenol broth. Medium here described is a type of eosin methylene blue agar. It contains: 1000 ml. distilled water; 10 grams peptone (Difco); 2 grams dipotassium phosphate; 10 grams lactose; 15 grams agar. Reaction is adjusted to pH 7.5. For use, following are added to 25 ml. of the agar: eosin, yellow, 2 percent, 0.5 ml.; methylene blue, 0.5 percent, 0.5 ml; phenol, 5 percent, 0.25 ml. Highly contaminated waters are plated directly in small quantities. In waters less densely polluted, bacteria must be concentrated. *B. coli* colonies are round, from 3 to 4 mm. in diameter. By transmitted light, center of colony is dark violet blue, covering at least three-fourths of its area, while periphery is bright gray blue, sometimes metallic in appearance. By reflected light, center of colony appears slightly elevated. Sub-surface colonies appear as small, dark blue lentils. *B. aërogenes* colonies are from 4 to 6 mm. in diameter, with deep brown centers when viewed by transmitted light. By reflected light they do not show metallic sheen. In case of doubt, colonies are picked and inoculated to peptone broth. —*G L. Kelso (Courtesy Chem. Abst.).*

Some Methods of Calibrating and Checking Filter Gages. Douglass Feben. Water Works Eng., 84: 4, 221, February 25, 1931. All mechanical appliances in filter plant should be checked periodically. All moving parts should be in good working order. Rate of flow gauge is usually checked by drop method. Influent valve is closed and either distance water on the filter drops in a given time is measured, or else time in which water drops a given distance. Two hooks at known vertical distance apart on filter wall are advantageous. Author describes in detail method used at Detroit wherein two electrodes connected to galvanometer serve to measure drop. Loss of head gauges can be best checked by open tube on effluent line; but it is necessary to check the indications throughout the range of the gauge. Author describes in detail unusual type of loss of head gauge in use at Detroit. Both special devices are illustrated with good dimension drawings.—*Lewis V. Carpenter.*

Determining Compensation for Land Taken by Condemnation. Leo T. Parker. Water Works Eng., 84: 5, 293, March 11, 1931. Right of eminent domain to corporations is based on public right and not on private interest. Value of property is determined true value and not amounts of reported offers. To establish market value by means of an offer, one must first establish fully the *bona fdes* of the offer. Sometimes when a portion of land has been taken in condemnation proceedings, courts have ruled that remainder of land has been damaged. Author cites number of special decisions arising from condemnation cases, namely, question of valuation between water company, or municipality, and property owner; members of board must act in joint meeting; condemnation proceedings not revokable; when official may obligate a municipality; and when right of city to acquire water plant held forfeited by delay.—*Lewis V. Carpenter.*

Proposed Colorado River Supply for Los Angeles Fraught With Difficulties. Frank E. Weymouth. Water Works Eng., 84: 4, 217, February 25, 1931. Only practicable water supply for southern California is Colorado river which, at its nearest point, is 210 miles from Los Angeles. Eleven cities have combined to form sanitary district and are making plans for aqueduct with ultimate capacity of 1500 second-feet, which is considered insufficient, but is maximum obtainable from this source. Pumping will be necessary, unless diversion above Boulder Dam is permitted, which would have the additional advantage of eliminating silt trouble. Black Canyon Line would have an initial lift of 1648 feet; power for pumping is to be obtained from Boulder Canyon. Study is being made of one possible location taking water from above Boulder Dam, impounding it in high reservoir, with 1000-mile aqueduct. Quality of water is expected to be good and numerous experiments have shown it to be suited to local agricultural requirements. At present time, topography has been taken over approximately 30,000 square miles of desert and mountain, preliminary plans have been drawn for number of different sources, and financial problems have been closely studied.—*Lewis V. Carpenter.*

NEW BOOKS

Scientific Survey of River, etc. From Report of Water Pollution Research Board, England, for 1929–30, 32 pp. Department of Scientific and Industrial Research. Comprehensive survey of typical river (Tees) flowing through industrial area is being made. In higher reaches, dissolved carbon dioxide and oxygen, nitrite, ammonia, chloride, hardness, pH value, and temperature are being determined, to ascertain changes by day and night; in tidal reaches, tar acids, cyanide, phosphate, iron, and opacity are determined in addition to the above; while influence of waterfalls upon aëration has been examined, and botanical and zoological results tabulated. The flood tide is strongest at below one fathom depth, the ebb is strongest at surface. Biological and chemical work on tidal reaches shows an area of polluted water with dissolved oxygen below normal, which probably accounts for small number of living forms. *Beet sugar effluents.* It is found that percolating filters can be operated satisfactorily for purification of pulp liquor at rate of 100 (Imp.) gallons per cubic yard of filtering material per day for liquor diluted to give a five days oxygen absorption value of 60 to 70 parts per 100,000. *Base exchange softening.* Investigation is being continued; effects of variation in temperature and in concentration of dissolved calcium, magnesium, and sodium salts are now being studied. Bacteriologically, process is without effect.—*W. G. Carey.*

A Study of the Pollution and Natural Purification of the Illinois River. II. The Plankton and Related Organisms. W. C. Purdy. Public Health Bulletin 198: U. S. Public Health Service, November, 1930, 212 pages. See also Public Health Reports, **46**: 3, 113, January 16, 1931. This valuable contribution brings out relation of plankton to various stages of natural purification of streams in general, but deals more particularly with conditions specific to the much discussed Illinois River, heavily polluted by sewage and stock yards waste from city of Chicago. Very briefly summarized, study indicates following changes as water progresses: (1) swift upper portion of the river, heavily polluted but thoroughly mixed, is well seeded initially with microscopic organisms from Des Plaines River and from lake Michigan; (2) gradually decreasing velocity determines gradual deposition of suspended matter over immense area of river bottom, facilitating biological action, (3) the grayish water becomes clear and loses its odor of sewage 70 to 80 miles down stream from Chicago Drainage Canal outlet; (4) correlated changes in plankton content are: (*a*) decrease of pollutional organisms initially predominant; (*b*) increase of organisms of the cleaner water class, these becoming and remaining predominant; and (*c*) increase in relative abundance of microscopic green plants; (5) in all sections of river and at all seasons, microscopic green plants were decidedly more abundant, volume for volume, than microscopic animals; (6) malodorous bottom sediments from the polluted upper reaches contained very large numbers of "sludge worms" and no gill-bearing insect larvae; whereas sediments from the lower reaches were free of odor, contained very few worms, and showed a variety of gill-breathing insect larvae. Bulletin is divided into seven chapters. Chapter I, very short, deals with remarkable

biological interest of Illinois River. Chapter II presents eleven abstracts of previous biological studies of the Illinois and of other streams. Chapter III describes the physiographic and pollutional features. Chapter IV gives object, scope, and limitations of study reported; also the field and laboratory methods employed. Chief objects were; (1) to ascertain distinctive features in the biology of successive zones; (2) to relate biological findings to other facts bearing upon pollution, i.e., time and distance factors, concentration of sewage, physical conditions in channel, etc.; and (3) to seek a better understanding of ecology of plankton forms characteristic of polluted water. Plankton, bottom sediments, and certain other samples were collected along practically entire length of river for period of 14 months. To secure adequate data, commensurate with scope of investigation, a reasonably large number of samples, properly distributed to show a year's régime, was imperative. Methods and procedures described and helpful glossary of terms will be found of especial value to investigators and students. Chapter V summarizes field observations and laboratory findings under the headings physiographic, physical evidence of pollution, biological reaction, plankton content, and bottom sediments. Chapters VI and VII discuss and summarize, seasonally, data according to sampling stations from upper and lower river respectively, concluding with general summary of conditions station by station, for each station. Appendices include: (A) Brief comparison of Illinois River with Ohio and Potomac Rivers. (B) Discussion of organisms other than plankton, and their probable significance. (C) Summary and classification of plankton findings at all stations. Photographs and charts supplement the 54 tables.—*R. E. Noble.*